# GEOTECHNICAL INVESTIGATION
## Uvas Creek Levee Recertification
## Gilroy, California

*Submitted to:*
**Schaaf & Wheeler, Santa Clara, California**

*Submitted by:*
**AMEC Geomatrix, Inc., Oakland, California**

June 2009

Project 13815.001

AMEC Geomatrix

June 22, 2009

Project 13815.001

Mr. Chuck Anderson
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Santa Clara, CA 95050

**Subject:** **Geotechnical Investigation**
Stevens Creek Levee Recertification
Gilroy, California

Dear Mr. Anderson:

AMEC Geomatrix, Inc. (AMEC) is pleased to submit this Geotechnical Investigation Report to support the Uvas Creek Levee Recertification project in Gilroy, California. This report was developed in accordance with our Agreement for Professional Services with Schaaf & Wheeler, dated April 15, 2008 (fully executed May 22, 2008). Our investigation included compiling and reviewing existing data, performing a field reconnaissance, performing field exploration, performing geotechnical laboratory testing, performing engineering evaluations and analyses, and preparing this report.

We conclude that we are not aware of any geotechnical considerations that we consider to stand in the way of recertification of the levee in accordance with 44 CFR 65.10.

If you have any questions about this report, please do not hesitate to call any of the undersigned. It has been a pleasure working with you and we look forward to working with you on other future phases of the project.

Sincerely yours,
AMEC Geomatrix, Inc.

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# GEOTECHNICAL INVESTIGATION REPORT
Uvas Creek Levee
FEMA Levee Certification
Gilroy, California

## 1.0 INTRODUCTION

AMEC Geomatrix, Inc. (AMEC, formerly known as Geomatrix Consultants, Inc. [Geomatrix]) is pleased to present the results of our geotechnical investigation for recertification of the existing levees on the northeast bank (i.e., the left bank when looking downstream) of Uvas Creek, between approximately 2,200 feet downstream of Thomas Road to the south, and 1,100 feet downstream of Santa Teresa Boulevard Bridge to the north, in the City of Gilroy, California. The location of the project is shown on the attached Site Location Map, Figure 1.

### 1.1 PURPOSE AND SCOPE

This investigation was performed on behalf of the Santa Clara Valley Water District (the District) and the City of Gilroy; our work was performed under subcontract to Schaaf & Wheeler, Consulting Civil Engineers (S&W). The purpose of our work was to evaluate the stability of the levees under existing, steady-state flood, rapid drawdown, and seismic loading, and to develop a professional opinion about whether the project levees meet the requirements of 44 CFR 65.10.

This study is part of a broader effort to evaluate the levees using the criteria set forth in 44 CFR 65.10 of the National Flood Insurance Program (NFIP), which includes minimum certification standards for levee design, operations, and maintenance. These standards must be maintained for the levees to remain accredited by FEMA as providing protection from the estimated 100-year flood (i.e., a flood with a return interval of 100 years). This report addresses the geotechnical aspects of the FEMA levee certification process; other engineering evaluations, such as hydrologic and hydraulic analyses to satisfy the FEMA criteria are not addressed in this report.

The scope of services to accomplish the above-stated purpose was outlined in the Professional Services Agreement (PSA) between AMEC and S&W, dated April 15, 2008 (fully executed May 22, 2008). Tasks performed by AMEC in support of this purpose have included the following:

1) Compile geotechnical data to describe surface and subsurface conditions, including the engineering properties of levee fill and foundation materials. AMEC's work was performed in coordination with a subsurface investigation and laboratory testing performed by Pacific Geotechnical Engineers (PGE), also under subcontract to S&W.

2) Perform geotechnical engineering analyses to evaluate liquefaction, seepage, embankment stability, and settlement in general accordance with the United States Army Corps of Engineers (USACE) EM 1110-2-1913. Specific analyses have included the following:

   a) Seepage through and beneath the levee
   b) Levee stability
   c) Levee settlement

3) Prepare this report to present the results of our analyses.

### 1.2 PROJECT TEAM

The project was directed by S&W as the prime consultant. Under subcontract to S&W, AMEC work included compiling and reviewing available pertinent geotechnical data, performing field reconnaissance, performing geotechnical engineering analyses, and preparing this report. Geotechnical borings and cone penetration tests (CPTs), as well as geotechnical laboratory tests were performed by Pacific Geotechnical Engineers (PGE), also under subcontract to S&W; their work was coordinated with S&W and AMEC. PGE's work will be presented under separate cover, but the boring logs, CPT logs, and laboratory test results are presented as appendices to this report.

### 1.3 REPORT ORGANIZATION

A project introduction, scope of work, and project team are described in Section 1.0. An overview of the levees is presented in Section 2.0. The methods of investigation are described in Section 3.0. The site conditions, including the regional geologic and seismic settings, are presented in Section 4.0. The history of the levees, based on a review of aerial photographs, is presented in Section 5.0. Surface and subsurface conditions, site geologic setting, groundwater conditions, and seismic site classification are described in Section 6.0. A discussion of our geotechnical analyses is presented in Section 7.0, and our conclusions are presented in Section 8.0. General limitations and references are presented in Sections 9.0 and 10.0, respectively.

Appendices present background data as well as supplemental documentation of our analyses.

## 2.0 LEVEE DESCRIPTION

The project levees lie along the northeast side (left bank) of Uvas Creek. As shown on Figure 1, the study area for this project follows Uvas Creek for approximately 11,400 feet (2.2 miles) between approximately 1,100 feet downstream of the Santa Teresa Boulevard Bridge extending downstream to approximately 2,200 feet downstream of the Thomas Road/Luchessa Avenue Bridge to the south. The project levees lie on the northeast side of the creek. Immediately upstream of the study area, Uvas Creek flows through a rural portion of southern Santa Clara Valley. Downstream of the study area, the creek continues for approximately 30 miles before reaching the open waters of Monterey Bay.

Development on the northeast side of the creek includes a high school and residential developments. Development on the southwest side of the creek includes a park, residential subdivisions, a middle school, and farmland.

Portions of the project levee were originally constructed prior to the 1969 air photos. By the late 1980s, the levees had been raised and widened by the District as part of a comprehensive grading and levee improvement project. In addition to constructing new levees, that improvement project also built the Thomas Road/Luchessa Avenue Bridge.

Aerial photographs of the levees with the District's stationing line are shown on Figures 2 through 5. Locations of features described in this report are referenced to the District's levee stationing system; however, the references to District stationing presented in this report are approximate.

Uvas Creek enters the reach of the current project where the northeast side levee begins, approximately 1,100 feet downstream of the Santa Teresa Boulevard Bridge, near Station 114+00. The creek exits the reach of the current project where the north side levee ends, approximately 2,200 feet downstream of the Thomas Road/Luchessa Avenue Bridge, near Station 0+00.

The crest of the levee is paved with asphalt from about Station 6+00 to 114+00, where it serves as an access road for levee maintenance and a pedestrian/bicycle trail. From about Station 0+00 to 6+00 the levee crest consists of a gravel maintenance road. A detailed description and photographs of the levee are provided in Appendix A.

Based on available data of cross sections and observations made during our field reconnaissance, existing levee slopes at the site appear to have maximum heights of approximately 15 feet on the waterside and 10 feet on the landside of the levee, with typical inclinations of about 2:1 (horizontal to vertical) and typical crest widths of about 14 ft.

We observed evidence of animal burrowing (apparently mostly gophers and possibly some ground squirrels) in a number of locations, as documented in Appendix A. We recommend regularly scheduled maintenance of the project levees to fill these holes and to remove the animals making them. Additional observations are presented in Appendix A.

A detailed review of levee pipe penetrations has been made by S&W in conjunction with this study. A summary of the major penetration is provided in Appendix A.

### 2.1 PAST LEVEE PERFORMANCE

It is our understanding that the project levees have performed quite well since construction, with no major erosion or sliding problems. It is our understanding that this reach of Uvas Creek has survived several significant floods with no major damage.

### 2.2 LEVEE PERFORMANCE DURING LOMA PRIETA EARTHQUAKE

Following the Loma Prieta Earthquake of October 17, 1989, the project levees as well as other levees in the region were inspected. We are not aware of any reported performance issues for the project levees that can unambiguously be attributed to the effects of the Loma Prieta Earthquake. However, the following reports have relevance to the current evaluation of levee stability, especially with regard to seismic stability:

1) A letter of January 30, 1989, from the Department of the Army to the District, indicated that the project levee construction had been completed as of January 6, 1989.

2) A Federal Emergency Management Agency Damage Survey Report was prepared for an inspection performed on January 22, 1990, for Llagas Creek levees located on the east side of Highway 101 and west of the intersection of the Llagas Creek with Bloomfield Avenue. The location is about 3 to 3½ miles to the east-southeast of the project levees. The report indicates there were three zones of levee that experienced settlement and cracks to 8 feet deep with a total levee length of about 160 feet. Poor-quality reproductions of site photos from that inspection that we reviewed were difficult to interpret, but it appeared the cracks were on the order of about 3 to 5 inches wide with vertical offset limited to less than about 1 inch.

3) A letter of September 12, 1990, from the USACE Maintenance and Planning Division to the USACE San Francisco District Engineer, presents the second semi-annual inspection report for the period of November 1, 1989 to April 30, 1990. It noted that the bike trail along the crest of the Uvas Creek Levee had several extensive longitudinal cracks in their previous report, but that the cracks had grown to extend over most of the bike trail. It is not clear from the report whether these cracks were a result of poor pavement construction, embankment compression and spreading, consolidation settlement of underlying clay foundation layers in response to the imposed embankment load, or small strains induced by the Loma Prieta Earthquake (which occurred just 2 weeks before the end of the first inspection period), or some combination of these and other factors. In fact, the report notes that the City of Gilroy contended that pavement was construction was inadequate,

although the USACE had not concurred as of the time of the letter. Of particular interest to our current work, it is difficult to rule out any of these factors as possible contributors to the cracking.

A copy of these records is presented in Appendix B.

## 3.0 SITE INVESTIGATION

Previous site investigations and laboratory testing has been performed by others, and the summary results are presented in Appendix C.

The site investigation performed for the project consisted of the following tasks:

- Data Review
- Field Reconnaissance
- Field Exploration (performed by PGE)
- Geotechnical Laboratory Testing (performed by PGE)

A summary of the site investigation methods are described in more detail below. PGE will provide a report under separate cover documenting the field and laboratory investigation.

### 3.1 DATA REVIEW

AMEC compiled and reviewed readily available geologic and geotechnical investigation reports concerning the project levees and contained in the files of the District. The primary information obtained during the data review includes previous borings by J.H. Kleinfelder & Associates (Kleinfelder, 1979), Anderson Geotechnical and Terratech for the U.S. Army Corps of Engineers (USACE, 1987), and Woodward-Clyde Consultants for Ruth and Going Inc. (Ruth, 1988) shown on a plan and profile sheet for the Uvas Creek project. Approximate exploration locations identified in previous reports are shown on Figures 2 through 5. Logs of these previous exploration locations and the results of relevant laboratory tests are provided in Appendix C.

In additional to the existing geologic/geotechnical investigation reports, AMEC reviewed published maps and reports to evaluate geologic and geotechnical data relevant to the project site, including readily available geologic and fault maps.

AMEC also reviewed 12 sets of historical black-and-white and color aerial photographs with a stereoscope to observe past land use and broad geomorphologic features at and around the project levees for information pertinent to the geologic/geotechnical conditions. Photographs taken in 1969, 1972, 1986, 1990, 1996, 1997, and 2008 had scales ranging from 1:6,000 to

1:36,000, and were supplied by and were reviewed at the offices of the Santa Clara Water District. Photographs taken in 1987, 1988, and 1989 had scales ranging form 1:7,200 to 1:36,000 and were supplied by and were reviewed at the office of Pacific Aerial Surveys. Summaries of the conditions observed on the photographs are provided in Section 5.0.

Detailed citations of these published and unpublished sources are compiled in the references section at the end of this report.

### 3.2 FIELD RECONNAISSANCE

On May 22 through May 23, 2008, and on May 27, 2008, AMEC performed field reconnaissance of the project levees to observe overall site and levee conditions, including levee geometry, and whether any obvious geologic/geotechnical concerns were exposed.

General reconnaissance activities included the following:

- Visually observing the levees, creek channel, and adjacent areas for evidence of settlement, erosion, ground deformation/cracking, and potential stability problems;
- Approximately measuring levee dimensions and slope inclinations;
- Identifying the presence and condition of erosion protection measures, outfalls, closures, floodwalls, and other structures/improvements along the levee alignments;
- Identifying trees, utilities, and other features that encroach on levee slopes;
- Observing water levels and flow conditions in the creek channel;
- Photographing the levees, adjacent areas, and other features of interest; and
- Locating utilities, penetrations, and other features of interest using handheld Global Positioning System (GPS) equipment.

Conditions observed during the field reconnaissance were documented in levee field reconnaissance logs, which also include brief descriptions of photographs and GPS locations collected during the reconnaissance effort. Findings of the field reconnaissance, including field logs, photographs, and GPS locations and feature descriptions, are presented in a site reconnaissance memorandum in Appendix A of this report.

### 3.3 FIELD EXPLORATION

A field exploration program, consisting of drilling exploratory borings and performing cone penetration tests (CPTs), was performed by PGE to explore the subsurface conditions on the project levees. The field exploration program is described in more detail in the following sections.

### 3.3.1 Exploratory Borings

Four borings (Borings B-1 through B-4) were drilled on the project levees on October 6 through October 8, 2008. The exploratory borings were advanced to depths of approximately 15 to 70 feet below the top of the levees at the approximate locations shown on Figures 2 through 5.

The borings were performed under the direction of PGE using either hollow-stem auger or mud-rotary wash drilling techniques. Subsurface materials were sampled using conventional 3-inch outside diameter modified-California (mod-Cal) and 2-inch outside diameter Standard Penetration (SPT) split-spoon samplers driven with a 140-pound auto-trip hammer dropping 30 inches in general conformance with ASTM D1586 procedures. The field exploration program is described in more detail in Appendix D. Logs of subsurface conditions encountered in the borings, prepared by PGE, are also presented in Appendix D.

### 3.3.2 Cone Penetration Tests (CPTs)

Ten CPT probes were used to explore the subsurface conditions at the site between September 29 and September 30, 2008. The CPT probes were 10 square centimeters and were advanced by John Sarmiento and Associates, of Orinda, California, working under subcontract to PGE, to depths of approximately 15 to 70 feet below the top of the levees. The approximate locations of the CPTs are shown on Figures 2 through 5. Graphic logs are presented in Appendix D.

## 3.4 GEOTECHNICAL LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. Samples were tested for dry density, moisture content, Atterberg limits, grain size distribution, undrained shear strength, and consolidation. The laboratory test program was prepared by PGE with input from AMEC. The laboratory tests were performed by PGE and by Cooper Testing Labs of Palo Alto, California, under subcontract to PGE. Tabular and graphic presentations of the test results are presented in Appendix E.

# 4.0 GEOLOGIC AND SEISMIC SETTING

## 4.1 REGIONAL GEOLOGY

The San Francisco Bay Area is located at the boundary between the Pacific and North American plates, two large crustal plates that are separated by the north-northwest-trending San Andreas Fault, within the California Coast Ranges Geomorphic Province. The geomorphology of the region includes parts of three prominent, northwest-trending geologic/geomorphic features, which include, from west to east the Santa Cruz Mountains,

Santa Clara Valley, and the Diablo Range. Santa Clara Valley forms part of an elongated structural block (the San Francisco Bay block) within the central Coast Ranges that contains San Francisco Bay and its surrounding alluvial margins (Page, 1989). This structural block is fault-bounded by the San Andreas Fault to the southwest and the Hayward-Calaveras fault zone to the northeast (see Figure 6).

Uvas Creek does not drain into the San Francisco Bay. Uvas Creek originates in the foothills on the eastern side of the Santa Cruz Mountains and flows southeast into the southern portion of the Santa Clara Valley. The creek then flows south, becoming Carnadero Creek south of California Highway 125, which flows south to join the Pajaro River. The Pajaro River then flows west through the Pajaro Gap to drain into the Watsonville Slough and Monterey Bay.

The oldest rocks in the region belong to the Franciscan Complex of Jurassic to Cretaceous age (205 to 65 million years before present [Ma]). These rocks are intensely deformed (i.e., folded, faulted, and fractured) due to ancient tectonic processes and, to a lesser extent, from more recent tectonic processes associated with the San Andreas Fault system. Franciscan rocks generally comprise the “basement” of the Coast Ranges northeast of the San Andreas Fault; Cretaceous granitic rocks, known as the Salinian block, comprise the basement of the ranges located southwest of the San Andreas Fault. A sequence of Tertiary (65 to 1.8 Ma) marine and nonmarine sedimentary rocks unconformably overlies, and locally is in fault contact with, the granitic and Franciscan basement rocks in the region.

Quaternary (1.6 Ma to present) surficial deposits are concentrated in the Santa Clara Valley and locally overlie the complexly deformed Cretaceous rocks and Tertiary strata in the adjacent hills. During the Plio-Pleistocene (5 Ma to 11,000 years ago [ka]) epochs, sediments eroded from the uplifting Diablo Range and the Santa Cruz Mountains formed broad alluvial fan complexes along the margins of Santa Clara Valley. The 5 Ma to 300,000 ka (Plio-Pleistocene) Santa Clara Formation, which consists of a sequence of fluvial and lucustrine sediments, was deposited unconformably on the older Tertiary and Franciscan rocks along the margins of Santa Clara Valley during this time and subsequently has been folded, faulted, and eroded. Although the deposition of the Santa Clara Formation spans a relatively long interval and the stratigraphy of the formation is complex, it has been used as a “marker horizon” for evaluating the recency of faulting in the Santa Clara Valley region. The Santa Clara Formation is unconformably overlain by younger Quaternary and Holocene (11 ka to present) alluvial and fluvial deposits (stream channel, overbank, and flood basin environments), which interfinger to the north with estuarine muds of San Francisco Bay (Helley et al., 1979).

South San Francisco Bay is a north-northwest-trending subsiding basin that is filled primarily with Quaternary alluvium (stream) deposits eroded from the surrounding margins and

estuarine (bay) muds. The Sangamon and Holocene bay muds are separated by the Quaternary alluvium and eolian (wind blown) sand deposits. Alluvium deposits consist of sediments eroded from the surrounding Santa Cruz Mountains and Diablo Range uplands. These alluvial sediments were transported and deposited by streams and include a mixture of sands, gravels, silts, and clays with highly variable permeability. In contrast, the fine-grained bay muds have very low permeability. The youngest Holocene bay muds underlie almost all of the original San Francisco Bay (Atwater et al., 1977; Helley et al., 1979), but do not extend into the southern portion of the Santa Clara Valley.

Due to movement on the San Andreas Fault and related faults, including the Hayward and Calaveras Faults, as well as the previous geologic history, a wide variety of igneous, metamorphic, and sedimentary rocks are present.

Geologic units are classified according to geologic time periods. The most recent time period is the Quaternary Period, which covers from 1.8 Ma to the present. The Quaternary Period is subdivided into the Pleistocene Epoch (1.8 Ma to 11 ka) and the Holocene Epoch (11 ka to the present). The Pleistocene includes a number of worldwide glacial periods of low sea level stands. Estuarine (bay) muds have been deposited in San Francisco Bay during high sea level periods of the Sangamon (70 ka to 130 ka) and the Holocene (less than 11 ka) (Atwater et al., 1977).

### 4.2 SEISMICITY

San Francisco Bay is considered to be one of the more seismically active regions in the world, based on its record of historic earthquakes and its position astride the San Andreas Fault system (see Figure 6). The San Andreas Fault system consists of several major right-lateral strike-slip faults in the region that define the boundary zone between the Pacific and North American tectonic plates. During the past 200 years, faults within this plate boundary zone have produced numerous small-magnitude and at least fifteen moderate to large (i.e., $M > 6$) earthquakes affecting the Bay Area (Toppozada et al., 1981; Ellsworth, 1990; Bakun, 1999). The U.S. Geological Survey (USGS) 2007 Working Group on California Earthquake Probabilities (WGCEP, 2008) estimated an approximately 63 percent probability that at least one major moment magnitude earthquake ($M_W \geq 6.7$) would occur in the San Francisco Bay Area before 2037, with a 21 percent probability that that such an earthquake will occur on the San Andreas Fault.

A large earthquake occurred in June 1838 on the Peninsula segment of the San Andreas Fault (Ellsworth, 1990). Toppozada and Borchardt (1998), Hall et al. (1999) and Hall et al. (2001), and Bakun (1999) reevaluated the data for this earthquake and estimated (local) magnitudes of 7.5, 7.0 to 7.4, and 6.8, respectively. Severe shaking of Modified Mercalli Intensity (MMI)

VIII to IX occurred in the vicinity of San Jose during this event, with houses shaken down and damage to walls of adobe structures (Toppozada and Borchardt, 1998). The MMI is an experience-based rating system used to categorize the intensity of shaking (prior to use of the seismograph), and is based on the relative damage done to structures/property and general uneasiness of the population as a result of the earthquake.

An earthquake having an estimated $M_L$ of 6.1 (Bakun, 1999) occurred in the San Jose region on November 26, 1858. This earthquake resulted in cracking of almost every brick, adobe, or concrete building in San Jose (Townley and Allen, 1939), corresponding to MMI VII to VIII (Toppozada et al., 1981).

A strong earthquake occurred on October 8, 1865, and apparently was centered in the Santa Cruz Mountains south of San Jose. This earthquake is estimated to have been of $M_L$ 6.5 (Bakun, 1999), and caused damage to buildings in San Francisco, Santa Clara, San Jose, and other areas (Townley and Allen, 1939). The reported damage from the 1865 earthquake corresponds to MMI VII to IX in the Santa Clara, New Almaden, and San Jose areas (Toppozada et al., 1981).

The 1868 Hayward earthquake occurred north of San Jose along the southern segment of the Hayward Fault and had an estimated magnitude of 7.0 (WGCEP, 2008). Surface rupture apparently extended from Oakland southward to the Warm Springs area of Fremont. The event reportedly resulted in damage or the complete destruction of every building in Hayward, as well as damage to buildings as far south as Gilroy (Townley and Allen, 1939). The reported damage in Santa Clara County corresponds to MMI VII to VIII.

During the $M_W$ 7.8 1906 San Francisco earthquake, the San Andreas Fault ruptured from Shelter Cove near Cape Mendocino southward to near San Juan Bautista. Maximum lateral displacements of 15 to 20 feet occurred north of the Golden Gate at Olema in Marin County (Lawson, 1908). Landslides, liquefaction, and ground settlement occurred throughout the San Francisco Bay region and in the vicinity of the surface rupture as result of this earthquake. Extensive damage occurred to many buildings in San Jose, corresponding to MMI VIII to IX (Toppozada and Parke, 1982). Youd and Hoose (1978) also report that ground failures resulting from the 1906 earthquake were locally observed in the coarse gravelly bottom of Coyote Creek.

More recent earthquakes in the region include the 1957 Daly City earthquake on the San Andreas Fault ($M_L$ 5.3); the Coyote Lake and Morgan Hill earthquakes of 1979 and 1984 on the Calaveras Fault ($M_L$ 5.9 and 6.1, respectively); the 1980 Livermore earthquake on the Greenville Fault ($M_L$ 5.8); and the 1989 $M_L$ 7.1 Loma Prieta earthquake on the San Andreas

Fault or a parallel subsidiary fault. Of these earthquakes, the strongest shaking and most damage resulted from the October 17, 1989, $M_L$ 7.1 Loma Prieta earthquake.

The 1989 earthquake ruptured on or southwest of the Santa Cruz Mountains segment of the San Andreas Fault and produced MMI VII effects in San Jose. No liquefaction or associated ground-failure effects related to the Loma Prieta earthquake were reported in the vicinity of the project levees by Tinsley et al. (1998). The closest published liquefaction or associated ground-failure effects related to the Loma Prieta earthquake were reported along the Pajaro River near Sargent, approximately 5.5 miles (8.8 km) south-southeast of the project levees, where sand boils and minor lateral spreading damaged a service road to a sugar beet loading facility (Tinsley et al., 1998) (but see Section 2.2 for discussion of unpublished ground deformation reports). Ground cracking and settlement were reported in the Sargent area in 1906 (Youd and Hoose, 1978).

The USGS prepared a Quaternary Geology (Sheet 1) and Liquefaction Susceptibility (Sheet 2) map pair that covers the area of the project levees; it indicates that the Holocene alluvial fan deposits (Qhf) that underlie the project levees have a moderate liquefaction susceptibility, and that the modern stream channel deposits (Qhc) (see Section 6.1) that fill the channel itself (but do not appear to underlie the project levees) have a very high liquefaction susceptibility (Knudsen et al., 2000, Sheet 2).

The California Geological Survey has not yet published Seismic Hazards Zones maps of the Gilroy and Chittenden Quadrangles, which will cover the project levees (these maps are in progress).

## 5.0 AERIAL PHOTOGRAPH REVIEW

A summary of the levee history, as interpreted from our aerial photograph review, is presented below. It should be noted that each summary only presents significant changes from each prior photograph date. The project levees are along the northeast side of the creek and the following observations are with respect to the northeast side of the creek.

### 5.1 10/28/1969 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch =500 feet provides complete stereo coverage. The creek is essentially in its present location. A low levee appears to be present between about Stations 108+00 and 45+00. Natural top-of-bank conditions appear to be present upstream of about Station 108+00. It is unclear if a levee is present downstream of about Station 45+00 except for a short section upstream of the Thomas Road crossing. Apparent spoil piles are present along the toe of the bank between about Stations105+00 and 99+00. Quarrying of the channel and bank deposits is present between about Stations 99+00 to

58+00. Mature trees are present along the channel downstream of about Station 58+00. A short section of levee is present just upstream of the Thomas Road crossing. The bank just upstream of the crossing is vertical and appears to be covered with concrete between about Stations 25+00 and 21+00.

### 5.2 10/25/1972 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 1000 feet provides complete stereo coverage. Downstream of the Thomas Road crossing, a levee may be present set back a short distance from the top of the bank. From about Station 33+00 to 26+00 a straight section of levee is present setback for a right bend. Quarrying of the channel and bank deposits is present between about Stations 81+00 to 61+00. Previous areas of quarrying appear to be abandoned and overgrown.

### 5.3 1/10/1986 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 500 feet provides no stereo coverage. A levee may be present upstream to about Station 114+00.

### 5.4 6/12/1987 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 600 feet provided complete stereo coverage. The existing levee between the Miller Avenue and Thomas Road has not been constructed.

### 5.5 11/5/1988 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 3000 feet provides complete stereo coverage. The existing levee is present and appears to be recently constructed. The Thomas Road Bridge is under construction.

### 5.6 6/23/1989 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 3000 feet provided complete stereo coverage. The existing levee is present and the construction of the Thomas Road overcrossing is complete.

### 5.7 4/25/1990 PHOTOGRAPHS

This included a set of oblique black-and–white and oblique color photographs of the recently reconstructed Thomas Road crossing and the banks upstream and down stream of the crossing, with conditions as existing.

### 5.8 2/12/1996 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 500 feet provided stereo coverage only between about Stations 106+00 and 92+00 in the area of the old quarrying. Along this section the levee is located at the edge at the top of the bank.

### 5.9 1/3/1997 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = ≈ 384 feet provides stereo coverage only up and down stream of the Miller Avenue crossing. The creek appears to be at high flow.

### 5.10 5/21/1997 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 500 feet provides stereo coverage only between about Stations 106+00 and 89+00.

### 5.11 2/28/2008 PHOTOGRAPHS

This set of photographs at a scale of ≈1 inch = 3000 feet provides complete stereo coverage. Conditions are as existing.

### 5.12 4/4/2008 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 2000 feet provides no stereo coverage. Conditions appear as existing.

## 6.0 LEVEE FILL AND FOUNDATION CONDITIONS

### 6.1 SITE GEOLOGIC CONDITIONS

Recent geologic maps covering the project levees by Dibblee (2005; 2006) indicate that the reach of Uvas Creek levees downstream of Santa Teresa Boulevard to US Highway 101 is underlain by Holocene alluvial deposits described as gravel, sand, and clay (Qa). The channel of Uvas Creek downstream of Santa Teresa Boulevard to US Highway 101 is mapped as underlain by Holocene alluvial deposits described as gravel and sand (Qg) representing stream channel deposits. On the south side of Uvas Creek between about levee Stations 54+00 to 37+00, remnants of an older, higher alluvial terrace extends to the channel. The terrace is mapped as underlain by older alluvial deposits described as pebble gravel and sand (Qoa) representing dissected Pleistocene alluvial deposits.

Bedrock on the northeast side of Uvas Creek upstream of Santa Teresa Boulevard consists of the Miocene Briones (?) Sandstone (Tbr), and Miocene Monterey Shale (Tm) (Dibblee, 2005). Bedrock on the southwest side of Uvas Creek consists of the Miocene Monterey Shale (Tm), Jurassic and Cretaceous Serpentine (sp), and Franciscan Greywacke Sandstone (fs) and

Greenstone (Dibblee, 2006). The northwest-southeast trending Carnadero fault is mapped along the edge of the valley south of Uvas Creek (Dibblee, 2006).

A recent surficial deposits map covering the area of the project levees (Knudsen et al., 2000, Sheet 1) indicates that the project levees are underlain by Holocene alluvial fan deposits (Qhf), and the channel is underlain by modern stream channel deposits (Qhc).

## 6.2 SUBSURFACE CONDITIONS

The results of the current and past subsurface exploration and laboratory test programs, together with the published geology (Section 6.1) and information gathered from the air photo review and site reconnaissance were used to develop subsurface stratigraphy and geotechnical properties for use in the analyses of levee seepage, stability, and settlement. The geotechnical materials encountered in the explorations consisted of Levee Fill over clayey alluvium underlain by sands and gravels. Each material is described below. The material properties uses for our analyses are discussed in Section 7.1.

A profile showing the levees crest elevation and interpreted subsurface stratigraphy is presented in Figures 7 and 8.

### 6.2.1 Levee Fill

The thickness of the levee fill encountered in the explorations varied somewhat along the length of the project levees; typically it was about 7 to 10 feet for most of the levee reach, but thinned to about 7 to 3 feet to the north of about Station 83+00. The fill generally consisted of stiff to hard lean clay (CL) and dense to very dense clayey sand with fine gravel (SC), with variable proportions of finer and coarser sand fractions.

The construction and improvement plans for the levee indicate that the fill was transported from a site south of the intersection of Miller Slough Creek and Llagas Creek.

### 6.2.2 Upper Clay

Upper native soils generally consisted of a clayey alluvial layer directly beneath the fill, typically with a thickness of approximately 3 to 30 feet. This layer consisted of generally soft to hard lean clay (CL) with variable amounts of fine sand, with scattered thin (several inches to a foot or two) zones of clayey sand.

### 6.2.3 Sand

Sand was encountered underlying the upper clay layer, soft to medium stiff clay layer, or gravel layer depending on reach cross section subsurface profile. This layer consisted of medium to very dense well-graded silty sands with variable amounts of gravel.

### 6.2.4 Soft to Medium Stiff Clay

Based on interpretation of the CPT soil behavior types, CPT-01 encountered a zone of soft to medium stiff clay sandwiched between sand layers in Reach A1. CPT-04 and CPT-05 encountered a similar zone of soft to medium stiff clay underlying the upper clay in Reach C. This clay layer had a lower undrained shear strength than was encountered in other CPTs or in any borings.

### 6.2.5 Gravel

Gravel was encountered underlying the upper clay layer in Reach D. This reach is located adjacent the old gravel quarry located between the levee and the creek channel. This material consisted of medium dense to dense well graded gravel with variable fractions of sand and silt.

### 6.2.6 Lower Clay

A lower clay layer was encountered underlying the sand layer in Reach B and E. The lower clay layer generally consisted of firm to stiff, low to medium plasticity clay with variable amounts of sand and gravel (CL).

## 6.3 GROUNDWATER CONDITIONS

Groundwater was encountered at a depth of approximately 55 ½ feet below the ground surface (bgs) in Boring B-1, approximately 38 feet bgs in Boring B-2, and approximately 19 feet bgs in Boring B-3. Because Boring B-4 terminates at 15 feet, the depth to groundwater could not be observed during drilling. Upon completion, all of the borings were backfilled to the ground surface with cement grout. It should be noted that groundwater observations in the hollow-stem auger borings may have been made before the passage of sufficient time to allow groundwater conditions to come to equilibrium. In addition, fluctuations in the groundwater level may occur due to variations in rainfall, temperature, and other factors not evident at the time the observations were made.

Groundwater levels in previous investigations ranged from about 8 to 25 feet below the native ground surface at the time the borings were drilled (Kleinfelder, 1979 and Ruth, 1988).

# 7.0 GEOTECHNICAL ANALYSES

The project levees were evaluated from a geotechnical standpoint in general conformance with the guidelines given in 44 CFR 65.10. The following analyses were performed on critical cross sections thought to represent the worst case scenario along the levee alignment:

1) The potential for liquefaction triggering was assessed for the embankment fill and underlying alluvial soil deposits during a 475-year return period earthquake event. These analyses were performed assuming what we consider to be a reasonably typical

groundwater conditions (i.e., groundwater located at about the normal year-round water level and not elevated by flooding) due to the low probability of the 475-year earthquake event occurring concurrently with periods of high water.

2) Seepage conditions through and under the levee embankments were evaluated using the finite element computer software SEEP/W (GeoStudio, 2007). Steady-state seepage conditions were very conservatively assumed to represent flood water level conditions; in other words, the calculation was performed as if the flood was in-place for an infinite period of time. Due to the short term duration of the 100-year flood events, a steady-state condition is unlikely to develop in the project levees.

3) The stability of the levee slopes was evaluated under both static and earthquake loading conditions using the limit equilibrium computer software Slope/W (GeoStudio, 2007). The static stability of the levee slopes was evaluated assuming a range of conditions, including:

   a) As-is conditions with no flood or seismic loading.

   b) Flood loading conditions, with the conservative assumption of steady-state seepage within the levees.

   c) Rapid drawdown, very conservatively assuming fully-developed and un-dissipated pore pressures.

   d) Seismic loading, assuming typical groundwater conditions, which is consistent with the USACE Manual EM 1110-2-1913.

4) Settlement analysis included the evaluation of both long term static settlement and seismically-induced settlement (e.g., liquefaction-induced settlement and seismic compression).

## 7.1 SELECTION OF CROSS SECTIONS FOR ANALYSES

### 7.1.1 Cross Section Stratigraphy

Our interpreted stratigraphy along the centerline of the levee is shown on Figures 7 and 8. The quantity and locations for slope stability cross sections were selected so that each section could reasonably be represented by a single subsurface stratigraphy for the purposes of the analyses. The stratigraphy for each cross section was developed to be representative of its location while being reasonably conservative in the inclusion of weaker zones encountered in the CPTs and borings.

The subsurface conditions encountered varied along the length of levee, but were reasonably consistent within each levee reach selected for analysis. As described in Section 6.2, the subsurface soils encountered during the subsurface exploration program generally consisted of embankment fill, underlain by alluvial soil deposits.

### 7.1.2 Cross Section Geometry

Based on a consideration of the variable subsurface conditions encountered during the subsurface exploration and the surface topography, six cross sections were selected for analyses, at locations shown on Figures 2 through 5 in plan view, and Figures 7 and 8 in profile.

We reviewed the as-built drawings and topographic cross sections provided by S&W at approximately 300-foot intervals along the project alignment. Within each reach that had been selected based on subsurface conditions, a cross section profile with the tallest and/or steepest slope was selected; this was considered generally to represent a conservative geometry with respect to slope stability.

### 7.1.3 Material Properties for Analyses

The primary material properties needed for analysis included unit weights, permeability, and strength. Properties selected for analysis are summarized on Table 1. These properties were selected as follows:

1) Unit weights for clayey zones were selected based on average values from field and laboratory data and consideration of published typical values. Unit weights for the sandy zones were selected based on a comparison published typical values for the type of material and calculated values based on laboratory water contents and an assumption of saturated conditions.

2) Permeabilities for clayey materials were selected based on published typical values for similar materials and engineering judgment. Permeabilities for granular materials were selected based on calculations using the Kozeny-Carmen equations and grain size distributions. Grain size distribution from laboratory tests was used where it was available, or estimated where laboratory test results were not available. We also considered published typical values for similar materials and engineering judgment.

3) Strengths for the embankment material were selected based on a consideration of laboratory strength tests, field pocket penetrometer tests, CPT results, and typical correlations with SPT values. In general, values for analyses were conservatively selected to envelop the lower bound of the data.

4) Strengths for the native clayey soils were also selected based on a comparison of laboratory strength tests, field pocket penetrometer tests, CPT results, and typical correlations with SPT values. In general, values for analyses were conservatively selected to envelop the lower bound of the data.

5) Strengths for the native sandy and gravelly soils were selected based on a comparison of laboratory results, and typical correlations with CPT and SPT values. In general, values selected for analyses were conservative.

6) In Reach A1 and Reach C, the conservative selection of undrained strengths following the approach used for other locations resulted in predictions of what seemed to be unrealistically-large seismic deformations. Further discussion of strength properties for these sections is presented below in Section 7.4.4.

The locations of the cross sections used for analyses are shown in Figures 2 through 5 as well as Figures 7 and 8. The cross sections are oriented looking downstream. In general, the crest width was about 14 feet, the slope heights are typically about 15 feet for the waterside of the levee and 10 feet for the landside of the levee, and the maximum slope inclinations are about 2:1 (horizontal to vertical).

### 7.2 LIQUEFACTION ASSESSMENT

Liquefaction is a soil behavior phenomenon in which a soil located below the groundwater surface loses a substantial amount of strength due to high excess pore-water pressure generated and accumulated during strong earthquake ground shaking. During earthquake ground shaking, induced cyclic shear creates a tendency in most loose to medium dense soils to densify by rearrangement of the soil-particle structure. The potential for excess pore-water pressure generation and strength loss associated with this volume change tendency is highly dependent on the density of the soil, with greater potential in looser soils. Recently-deposited (i.e., geologically young) and relatively loose natural soils, and uncompacted or poorly compacted artificial fills, located below the groundwater table, are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible. Silty and clayey soils tend to be less susceptible than sandy soils to liquefaction-type behaviors. Even within sandy soils, the presence of finer-grained materials reduces susceptibility. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clayey soils and bedrock generally are generally not susceptible to liquefaction.

Permanent ground displacements due to lateral spreads or flow slides and differential settlement are commonly considered significant potential hazards associated with liquefaction. These displacement hazards are direct products of the soil behavior phenomena (i.e., high pore water pressure and significant strength reduction) produced by the liquefaction process. Lateral spreads are ground failure phenomena that occur near abrupt topographic features (i.e., free-faces such as creek banks) and on gently sloping ground underlain by liquefied soil. Lateral spreading movements may be on the order of inches to several feet or more and are typically accompanied by surface fissures and slumping.

Earthquake-induced settlement occurs with the dissipation of excess pore pressure generated by ground shaking that, as described above, is associated with the tendency for loose, saturated soils to rearrange into a denser configuration during shaking. Such dissipation will

produce volume decreases (termed consolidation or compaction) within the soils that are manifested at the ground surface as settlement.

Saturated granular materials, potentially susceptible to liquefaction from strong earthquake shaking, were encountered at scattered locations in the borings and CPTs, and these layers were generally relatively thin.

### 7.2.1 Liquefaction Analysis Methodology

Ground motions (i.e., Peak Ground Acceleration, or PGA) were estimated from the California Geologic Survey (CGS) Probabilistic Hazard Assessment Model, (CGS, 2003) for a 475-year return period earthquake.

The deaggregated contribution to the probabilistic seismic hazard was estimated from the U.S. Geological Survey (USGS, 2002), 2002 interactive deaggregations web site.

Liquefaction susceptibility was assessed using the software program CLiq v1.3 (GeoLogismiki, 2006). CLiq takes measured field CPT data and assesses liquefaction potential, and post-earthquake vertical settlement given a user-defined earthquake magnitude and PGA. CLiq utilizes the NCEER method (Youd, et al., 2001) along with calibrated procedures for post-earthquake settlements and lateral displacements by Zhang et al. (2002 and 2004).

It may be noted that CPT correlations underestimate the density and strength of sands that are located near to clays; this effect is more pronounced in proportion to the softness of the clay and when the sand is sandwiched between clay layers. CLiq is not able to correct for this “thin layer” effect, so the liquefaction potential, liquefaction-induced settlement, and strength loss for thin sandy layers tend to be over-estimated in the CLiq plots shown in Appendix F.

### 7.2.2 Results of the Liquefaction Analysis

The PGA for an alluvial site in the vicinity of Uvas Creek was estimated to be 0.61g, for a 475-year return period (CGS, 2003).

An earthquake magnitude of 7.9, corresponding to the modal magnitude of the deaggregated probabilistic motions (USGS, 2002), was used for liquefaction analysis.

Results of the liquefaction analyses indicate that the sandy soil layers encountered would likely liquefy during an earthquake event with a 475-year return period. A maximum permanent settlement of 2.5 inches (CPT-01) was estimated to result from liquefaction during an earthquake event with a 475-year return period. It may be noted that many of the explorations conducted for this study encountered thinner zones of potentially liquefiable sandy soil

material. Some explorations conducted for this study did not encounter any zones of potentially liquefiable soils. Liquefaction analyses output are included in Appendix F.

### 7.2.3 Liquefaction-Induced Lateral Spreading

We considered the possibility of liquefaction-induced lateral spreading. However, as indicated by Youd et al. (2003), lateral spreading is not likely to develop with corrected blow counts above 15. Most of the corrected blow counts for liquefiable material were greater than 15, and those that did not appear to be influenced by softer clay layers above and/or below the sand layer such that a thin-layer correction would increase the effective corrected blow count above 15. Therefore, it is our opinion that a liquefaction-induced spreading is not likely to be a significant effect for the project levees.

## 7.3 SEEPAGE ANALYSIS

Water, being driven by a large head differential, moving through or under an embankment can cause increased seepage uplift forces in the embankment and/or the embankment foundation. Under adverse seepage conditions, if the weight and cohesiveness of the embankment and/or foundation materials is not sufficient to restrain the seepage uplift forces acting on them, the materials have the potential for piping (erosion), which could lead to a loss of material and/or strength, thus impacting levee performance. It is therefore appropriate to evaluate the potential for seepage during flood conditions on the landside of levees.

Under normal flow conditions, water in Uvas Creek flows in a channel below the elevation of the toes of the levees. During flood conditions, water can rise temporarily against the waterside of the levees. During a 100-year flood the waterside slope is expected to experiences water as high as about 4 feet below the levee crest. The high water provided by S&W for a 100-year flood is shown in profile on Figures 7 and 8.

One representative 100-year flood hydrograph is presented in Appendix G. As can be seen in the hydrograph, the duration of high water is only a matter of a few hours. Based on the short duration of the flood stage, it is unlikely that seepage will extend more than a few feet into the levee; however, as is a common and conservative practice with levee evaluations, we preformed stability calculations based on the assumption of steady-state seepage, which yields conservative stability results that bounds the worst case scenario; actual impact of seepage on stability will be less than calculated.

As a simple and conservative evaluation of the potential for internal erosion or piping to occur, it is common to check that the maximum average exit gradient at the landside toe is less than 0.5 in a steady-state condition. The results of this check are discussed below in Section 7.4.2.

The computer program SEEP/W (GeoSlope, Ltd., 2007) is a two-dimensional, finite-element seepage analysis program. It was used to estimate the steady-state pore pressures within and underlying the levee. The resulting pore pressures were imported into SLOPE/W (GeoSlope, Ltd., 2007) for slope stability analysis. The program also calculated steady-state exit gradients discussed below. The representations of the surface and subsurface conditions for seepage analyses of Cross Sections A1 through Reach E are provided on Figures 9-1 through 9-6, respectively. The methodologies for determining the initial conditions and hydraulic conductivity for the analyses are explained below.

### 7.3.1 Initial Conditions

The initial groundwater conditions were estimated from historical boring data, data from the subsurface exploration program, and judgment regarding likely gradients associated with Uvas Creek. During drilling, the observed groundwater elevations varied from 200 feet (toward the southwest end of the project) to 160 feet (toward the northeast) (NAVD 88), based on available water levels from the previous and current subsurface investigations. We assumed that stream water levels were about 4 feet above the channel invert, which appears generally consistent with LIDAR data except at Cross Section A1 where the channel is narrower and appears to have approximately 6 feet of water above the channel invert. For our stability analyses, the groundwater elevation was also assumed to be about 4 feet above the invert of the channel.

Maximum flood stages were also provided by S&W and reproduced in Appendix G; these values are also shown on Figures 7 and 8. For our analyses, the flood level water level was simplified to be about 4 feet below the levee crest, which is within about 1 foot of the elevations provided by S&W.

### 7.3.2 Boundary Conditions

Permeabilities of soil materials that are within the saturated zone are estimated using various methods described in Section 7.3.3. For this analysis, general conductivity functions were chosen based on type of soils encountered in the subsurface explorations. The following boundary conditions were applied to the model for the flood-stage seepage analyses:

- A fixed total head boundary condition, corresponding to the analyzed water surface elevation, was applied along the waterside slope and channel.
- On the landside, a potential seepage face boundary (nodes that allow water to flow out of the model boundary) was applied and extended to the landside boundary.
- On the bottom of the model and vertical boundary along the centerline of the channel, a no-flow boundary was applied.

- The landside vertical drawing extents had a total head boundary condition with an elevation equal to the elevation at the drawing extents. This and the potential seepage face boundary allowed SEEP/W to estimate a phreatic surface through the levee.

Results of the steady-state seepage analyses are shown in graphical form in Figures 9-1 through 9-6, and Table 2 shows the approximate maximum calculated exit gradients from the seepage analyses.

### 7.3.3 Hydraulic Conductivity

During the subsurface exploration program, soil samples of the levee fill and alluvial soils were retrieved and grain size distributions were performed on select samples.

From the grain size distribution of the sandy and gravelly material, the vertical permeability, $k_v$, was estimated using the Kozeny-Carman equations (Carrier, 2003). From the higher bound average of all sand grain size distributions for this layer the estimated saturated hydraulic conductivity is about 9 x $10^{-2}$ cm/s. Similarly, the average estimate of the saturated hydraulic conductivity of the gravel is about 2 x $10^{-1}$ cm/s. These values are on the high end of the calculation and are conservative to account for the variability in the soil types encountered during the field exploration program. Due to depositional anisotropy, we assumed the horizontal permeability, $k_h$, of about 30 x $10^{-2}$ cm/s (i.e., $k_h / k_v \sim 4$) for the sand and $k_h$, of about 20 x $10^{-1}$ cm/s (i.e., $k_h / k_v \sim 10$) for the gravel.

All the clayey layers were assumed to have the same permeability. Permeability was estimated using Holtz and Kovacs (1981) and engineering judgment. For the clay an average saturated vertical hydraulic conductivity, $k_v$, on the order of 1 x $10^{-6}$ cm/s was assume for the clay layers. We assumed a ratio of $k_h / k_v \sim 10$ for the alluvial clays (assuming there will likely be siltier or sandier strata that will influence horizontal permeability).

The levee fill the soil type was assumed to range from very fine sand to a mixture of sand, silt, and clay with stratified clay deposits. From Holtz and Kovacs (1981), a vertical hydraulic conductivity of about 1 x $10^{-4}$ cm/s was assumed. We assumed a ratio of $k_h / k_v \sim 4$ for the levee fill.

### 7.3.4 Steady-State Seepage Analysis Results

Figures 9-1 through 9-6 show the contours of pressure head output from SEEP/W for Cross Sections A1 through E. The reported maximum exit gradient for each cross section was taken from the largest gradient contour near the landside toe.

The maximum exit gradient for each section is summarized in Table 2 for the hypothetical steady-state condition. Even with a steady-state condition conservatively assumed, the exit gradient is expected to be below 0.5 for all reaches.

## 7.4 SLOPE STABILITY ANALYSIS

The computer program SLOPE/W (GeoSlope, Ltd., 2007) utilizes two-dimensional, force and moment limit-equilibrium methods to calculate factors of safety along specified slip surfaces through an embankment. Spencer's method, which satisfies both force and moment equilibrium and assumes a constant angle of inter-slice force, was used for all analyses. By specifying material zones delineated by strength functions, SLOPE/W estimates the normal stresses acting on a potential slip surface to calculate the shear strength along the surface and compares the strength with estimated (gravity-induced) driving stresses to calculate a factor of safety.

The same cross sections and soil profiles selected for the seepage analysis were used for the slope stability analysis. Our levee stability analysis generally followed the procedure described in USACE EM 1110-2-1913. Four loading conditions were considered for each of the six cross sections:

- Existing Condition
- Steady-State Seepage Condition
- Rapid Drawdown Condition
- Earthquake Condition

These conditions are discussed below.

### 7.4.1 Existing Condition

The existing condition considers a situation where the water level in the creek and ground water level are as described above in Section 7.3.1. The analyses of slope stability for the existing condition were performed using effective stress strength parameters.

Twelve cases were analyzed, including both waterside and landside of all cross sections. The factors of safety are summarized in Table 2. The factors of safety of levee slope stability analyzed for the existing condition are satisfactory, ranging from 2.0 to 5.9. Figures 10-1 through 10-12 show the configuration for each case, the critical failure surface, the minimum factor of safety, and contours of factor of safety.

### 7.4.2 Steady-State Seepage Condition

The steady-state seepage condition considers a situation where the flood water level is sustained long enough to create steady-state flow through the levee, increasing pore pressures within the levee and foundation materials, thereby decreasing the strength of the levee and levee foundation materials. We note that the steady-state seepage assumption, though consistent with common practice, is conservative for the levee and levee foundation conditions identified for our study where the flood hydrograph has a relatively short duration.

Locations of phreatic surfaces were taken directly from the results of the steady-state conditions developed in the seepage analysis using the design flood level. As defined in the seepage analysis, the design flood level is approximately at 4 ft below the levee crest. The stability computations were performed using effective stress strength parameters. Due to the water pressure against the waterside slope of the levee, the waterside of the levee is deemed more stable than existing condition. Therefore, only the landward side of cross sections A1 through E was analyzed for the slope stability under steady-state conditions.

The factors of safety of these six cases are summarized in Table 2. The results indicate that the levees under steady-state seepage condition are stable, with factors of safety ranging from 3.2 to 4.6. Figures 11-1 through 11-6 show the configuration for each case, the critical failure surface, the minimum factor of safety, and contours of factor of safety.

### 7.4.3 Rapid Drawdown Condition

The rapid drawdown loading condition considers a situation where flood water level are sustained long enough to create steady-state flow through the levee, and then water levels are quickly lowered, reducing the buttressing effect of the water loads on the waterside slope, thereby increasing the driving stresses on the potential failure surface. It was conservatively assumed that drawdown would be sufficiently rapid that no drainage would occur in materials with low permeability. Effective stress strength parameters were used with a conservatively-selected phreatic surface located at the ground surface on the waterside slope up to the elevation of the maximum flood level, and within the levee the steady state-seepage phreatic surface was used.

Only the watersides of the levees were analyzed for the six cross sections for rapid drawdown condition. The landsides of the levees will not be less stable than for the steady-state seepage condition, and therefore these slopes were not analyzed for rapid drawdown conditions.

The factors of safety of these six cases are summarized in Table 2. The results show that the levee under rapid drawdown condition is stable, with factors of safety ranging from 1.3 to 3.5.

Figures 12-1 through 12-6 show the configuration for each case, the critical failure surface, the minimum factor of safety, and contours of factor of safety.

### 7.4.4 Seismic Stability Analysis

SLOPE/W provides an option to specify the resisting shear stress along the potential sliding surface when performing a pseudo-static analysis with an applied seismic coefficient. In this case, SLOPE/W first performs an analysis for a particular slip surface without applying the seismic force, and computes the shear resistance at the base of each slice. Next, the shear resistance is kept constant based on the initial confining stress prior to application of the seismic force. Then, the analysis is repeated for the same slip surface, but including the seismic force. This option (where the undrained strength during earthquake loading is specified based on the confining stress prior to the earthquake) was used in this study.

#### *7.4.4.1 Initial Seismic Screening*

The seismic stability of the levees was evaluated in two stages. For the initial stage we performed a pseudo-static seismic screening in which a seismic coefficient of 0.15g was applied, as suggested by Seed (1979) for a magnitude 8.25 earthquake. We conservatively reduced the undrained strength by 20 percent for soils that were thought to have a potential for seismic softening, as suggested by Makdisi and Seed (1978). Seed indicates that slopes given a seismic coefficient of 0.15g and a resulting pseudo-static safety factor of at least 1.15 for full strengths or at least 1.00 for reduced strengths will have deformations less than about 3 feet.

For this initial screening, computed factors of safety for both waterside and landside of the levee were in the range of 1.13 to 3.05 (see Table 2). These screening cases are presented on Figures 13-1 through 13-12. The pseudo-static screening analyses of slope stability indicate that the slopes of the levees have factors of safety greater than the minimum value 1.00 for reduced strengths (see Table 2). Although these screening results suggest that the permanent displacements induced by the earthquake will not be larger than about 3 feet (Seed 1979), some were only marginally above the typical screening criteria and so additional evaluation was considered warranted as described in the following text.

#### *7.4.4.2 Seismic Deformation Analysis*

For this second stage of analysis, permanent deformations were estimated using the yield acceleration concept proposed by Newmark (1965) as modified by Makdisi and Seed (1978). The results of these additional runs are presented on Figures 14-1 through 14-12.

The procedure used to estimate permanent deformation included the following steps.

- A yield acceleration, ky, at which a potential sliding surface would develop a factor of safety of 1.0, was determined using limit-equilibrium, pseudo-static slope stability methods. The yield acceleration depends on slope geometry, phreatic surface conditions, undrained shear strength of the slope material (and/or the residual strength of the liquefied zones), and the location of the potential sliding surface. The soil strength properties were reduced by 20% from the static undrained conditions (except as discussed below for Sections A1 and C), therefore the yield accelerations found in these analyses are considered to be conservative.

- The peak ground acceleration for a 475-year return period in alluvium was estimated from the CGS Probabilistic Seismic Hazards Map, 2002, to be 0.61g. With relatively low levees such as those along Uvas Creek, there is very little amplification of ground motion from the foundation to the crest of the levees; conversely, averaged accelerations within a sliding mass will not be much less than the PGA. Therefore we consider it appropriate to use the PGA directly for the seismic coefficient.

- For a specified potential sliding mass, the seismic coefficient time history of that mass was compared with the yield acceleration, ky. When the seismic coefficient exceeds the yield acceleration, down slope movement will occur along the direction of the assumed failure plane. The movement will decelerate and will stop after the level of induced acceleration drops below the yield acceleration, and the relative velocity of the sliding mass will drop to zero. The magnitude of the accumulated permanent displacement can be calculated by double-integrating the seismic coefficient time history where it exceeds the yield acceleration. However, to simplify this calculation, it is normal to use simplified design charts developed by Makdisi and Seed (1978) that relate the yield acceleration, the peak horizontal acceleration, and the predicted earthquake magnitude to the anticipated slope deformation.

The yield acceleration, ky, was calculated using limit-equilibrium stability analyses, as described above. The estimated ky values, factors of safety, and expected deformations for based on this analysis for the waterside and landside slopes of Cross Sections A, B, D, and E are summarized in Table 2. The computed lateral deformations are from about 1 to 38 inches. This means that, under seismic loading such as could occur during a major earthquake on a nearby fault, portions of these slopes may be expected to deform somewhat. This would result in some loss of freeboard, with the magnitude of freeboard loss being on the order of one third to one half of the anticipated lateral deformations. In addition, some longitudinal cracking of the embankment should be expected, and some cross-levee cracking could occur.

Because of the long return period for a major earthquake, it is unlikely that a flood will occur concurrently with a major earthquake. Similarly it is also unlikely that a major flood (i.e., with a long return interval) would occur in the months following the earthquake.

#### *7.4.4.3 Seismic Deformation Analysis for Cross Sections A1 and C*

For Cross Sections A1 and C, the deformations calculated using strengths that were developed in following the conservative approach described above resulted in predictions of

deformations that were quite large. This was caused in significant part by the presence of a soft to medium stiff clay layer in each of these Cross Section reaches. In order to check the reasonableness of the initial predicted deformation values, we performed a back check of the deformations that would be predicted for the Loma Prieta Earthquake using these conservatively-developed strengths. From the semi-annual inspection reports discussed in Section 2.2 we know that the actual Loma Prieta deformations were less than a couple of inches and may have been essentially zero.

The California Strong Motion Instrumentation Program (CSMIP) time history records from the Loma Prieta Earthquake are published by the California Division of Mines and Geology report (Shakal et al., 1989). Near the project site, two sets of time histories have locations underlain by alluvial fan or terrace deposits (the project levees are underlain by alluvial fan deposits) and are within about 1 to 1½ miles of the site. One additional set of time histories was recorded nearby, but that site was underlain by bedrock and so this recording was considered less-representative of the conditions at the site.

The selected time history sets were recorded with two orthogonal horizontal components and a vertical component. As published in the Pacific Earthquake Engineering Research Center (PEER) database (PEER 2005), the horizontal components had been rotated to fault-normal and fault parallel orientations, and these directions were reported with the record metadata. We subsequently rotated these processed records to correspond to a direction roughly normal to Cross Sections A1 and C.

In these rotated time histories we then identified the PGA in the direction normal to the levee slopes. Using these PGAs, we then performed Makdisi-Seed estimates of predicted deformation considering the yield coefficients calculated from the analyses described above. The resulting deformation predictions were in the range of 8 to 20 inches for the Loma Prieta Earthquake. Because the observed Loma Prieta deformation of these levee sections was less than this conservatively-calculated range, we inferred that the degree of conservatism in estimating the shear strength for the soft to medium stiff layers was unnecessarily and inappropriately large.

In response, we revised our analyses for these levee sections as follows:

- The material strengths were adjusted to match actual values measured within each reach, rather than using a conservative strength envelope.
- For the fill, upper clay, and softer clay, the revised strengths were based largely on the varying undrained shear strengths developed from CPT interpretations adjusted in light of laboratory test and SPT results.

- For the sands, the strengths were not modified.

- Because the sizes of the critical sliding masses were fairly large and deep, we considered that the average acceleration within the slide mass would be less than the PGA due to phase shifting and wave incoherence. For simplicity, we used a reduction factor $r_d$ originally developed for use with liquefaction analyses.

- A block type failure was used to allow the critical failure surface to pass along the softer clay layer. The block type failure gave a lower factor of safety when compared to the circle method for these analyses due to the strength and location of the soft to medium stiff clay layer.

With these modifications, we were able to predict deformations in response to the Loma Prieta Earthquake that were in the range of 1 to 4 inches, which we consider to be within a reasonable range of the reportedly observed performance. Given the satisfactory results of the back calibration, we then used these same modifications to develop deformation predictions for the design-level earthquake. This resulted in the predictions presented in Table 2 for Sections A1 and C, with a range of about 2 to 49 inches.

## 7.5 SETTLEMENT EVALUATION

### 7.5.1 Static Settlement

Based on a comparison of design elevations (corrected to NAVD88) and recent survey elevations (NAVD88) of the project levee, the recent survey elevations for most sections of the levee are either at or above the design elevations, with a few exceptions noted below.

- The cross section at levee Station 52+39 has a crest elevation 7 inches below design crest elevation. This area has approximately 20 foot thick layer of clay that may have consolidated since construction in 1989.

- The area near Cross Section 60+54 is approximately 5 inches below design crest elevation. This levee section is underlain by a layer of clay that is about 10 feet thick and that likely has consolidated since levee construction.

- The design elevation of the levee from 110+46 to 114+00 is higher, by about 1 ½ feet, than the recent survey data. From the design documents, the limit of construction of the levee is 105+00; therefore no levee was constructed between 105+00 and 114+00. Based on the field reconnaissance log, this does not appear to constitute levee because the ground surface of both waterside and landside are approximately the same elevation as the levee crest.

In our opinion it is likely that these clay layers are over-consolidated and that post-construction settlement is a result of re-compression of the clay layer. Because recompression normally occurs relatively rapidly, we expect that this recompression would have been essentially completed no more than a few years following construction. In our opinion, it is likely that future consolidation-related settlements will be significantly less than past settlements.

### 7.5.2 Seismic Settlement

As described above in Section 7.2, earthquake-induced settlement is expected to be on the order of 1 to 3 inches due to densification of underlying sand layers.

## 8.0 CONCLUSIONS

Based on the results of our geotechnical investigation, it is our opinion that the project levees Uvas Creek Levees meet the geotechnical criteria set forth in 44 CFR Section 65.10 of the National Flood Insurance Program and will remain reasonably stable under the loading conditions described in USACE Design Manual EM 1110-2-1913. A summary of each of the geotechnical criteria is provided below.

### 8.1 EROSION OF LEVEE EMBANKMENT DURING BASE FLOOD

It is our understanding that the levee has experienced moderately-high flood waters in the past. The levee slopes generally consist of clayey soils that are generally well vegetated with grasses so that they are expected to be generally resistant to erosion. In addition, we are not aware of any significant erosion damage caused by past flood flows, with three exceptions noted below:

- It is our understanding that there has been erosion to the bank area beneath and upstream of the Thomas Road Bridge in the past, but that this area has been repaired and protected with riprap. Since the repair, it appears that the slopes have been stable and have not experienced further erosion.
- Downstream of Thomas Road Bridge, erosion of the stream bank is present between Stations 14+50 and 17+50. The levee is set back approximately 100 feet from the channel; therefore the erosion in this area has more of an impact on the bicycle trail about 50 feet from the channel rather than the overall stability of the levee.
- One area of creek bank between Stations 42+00 to 47+00 has experienced recent erosion and some bank instability. However, the current levee has been set back nearly 50 feet from this erosion feature such that this is little current threat to the levee.

We recommend that the project levees be inspected on a regular basis for any signs of increased erosion. In addition, the levees should be inspected for signs of erosion following any major flood events.

### 8.2 LEVEE EMBANKMENT STABILITY

#### 8.2.1 Static Stability

Detailed analyses were performed to evaluate the embankment stability for various loading conditions. The loading conditions considered during this study included: (1) existing conditions; (2) steady-state seepage; (3) earthquake loading from a 475-year event, and (4)

rapid drawdown. For each static loading condition, the calculated minimum Factors of Safety met or exceeded the stability criteria as described in USACE Manual EM 1110-2-1913 Design and Construction of Levees.

### 8.2.2 Seismic Stability and Deformation

As presented in Section 7.4.4, we anticipate that a major earthquake could result in up to about 4 feet of seismic deformation to the project levees. Due to the conservatisms in our analyses, we anticipate that most of the project levees will move less than 1 to 3 feet, and that the higher end of this range is likely to occur only in localized areas. We also anticipate that the strains would be distributed over a broad shear zone. Based on geometric considerations, we anticipate that the amount of loss of freeboard would be about one third to one half of the lateral deformation, i.e., on the order of up to 1 to 2 feet.

As noted above it is unlikely that a large flood will occur during or shortly after a large earthquake. In addition, the existing freeboard is such that flood protection would likely remain even following a major earthquake, albeit with reduced freeboard. Due to the potential for structural damage to occur to the levees during seismic deformation (e.g., if levee deformation occurs, then longitudinal cracks are likely to develop and cross-levee cracks may develop), the levees should be inspected following any major earthquake, and it should be anticipated that some repair work will be needed.

## 8.3 LEVEE FOUNDATION STABILITY

Analyses were performed to evaluate the static and seismic settlement potential of the project levees. Based on these results, we estimate future static settlement of less than a couple of inches. In addition, we estimate approximately 1 to 3 inches of settlement during a 475-year earthquake event due to seismic densification.

Although localized zones of the foundation may be subject to liquefaction, we calculate that the liquefied strengths will generally be similar to the conservative clay strengths we assumed for our analyses, so the stability will not be greatly affected by these local occurrences of liquefaction.

## 8.4 ENCROACHMENTS/MAINTENANCE

During our field inspections, we observed several minor encroachments, such as trees and shrubs growing in the waterside and landside of the levee slope, as documented in Appendix A. The impact of the presence of these encroachments on the levee stability is considered to be small and they were not incorporated into our stability analyses; in our opinion they will not decrease the calculated Factor of Safety below the stability criteria established in USACE Manual EM 1110-2-1913 Design and Construction of Levees.

There was one pipe penetrations of a recycled water line extending into the Uvas Creek channel from beyond the levees. We did not observe evidence of geotechnical problems, such as seepage along these penetrations. However, the control gate should be monitored and maintained as appropriate to keep them functioning properly.

Furthermore, we observed vegetation and evidence of animal burrowing (mostly gophers and possibly some ground squirrels) in a number of locations, as documented in Appendix A. We recommend regularly scheduled maintenance of the project levees to fill these holes and to remove the animals making them.

## 9.0 LIMITATIONS

In the performance of its professional services, AMEC, its employees, and its agents comply with the standards of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. No other warranty, either express or implied, is made or intended in connection with the work performed by us, or by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings. In the event that conclusions or recommendations based on data in this report are made by others, such conclusions and recommendations are not our responsibility unless we have been given an opportunity to review, and we concur in writing with, such conclusions or recommendations.

In presenting data generated or collected by others, AMEC does not warrant or in any other way attest to the accuracy or applicability of the data. Locations of previous explorations by others have been approximated based upon figures provided in reproduced reports, and in many cases do not provide sufficient information or detail for accurate or precise locating of the explorations. As such, locations should be considered approximate.

The scope of work for this study did not include an environmental assessment or investigation for the presence of natural or man-made contaminants in the soil, bedrock, or groundwater. No inference of such conditions should be made from information contained in this report.

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# TABLES

**TABLE 1: MATERIAL PROPERTIES**

| Uvas Creek Soil Properties | | | | | | | |
|---|---|---|---|---|---|---|---|
| Soil Type | Properties | | Existing | Steady-state | Drawdown | Seismic | Source |
| Fill (CL/SC) | Unit Weight, $\gamma_t$ | 126 pcf | | | | | Average of all borings in fill material |
| | Permeability, $k_v$ | $1.0 \times 10^{-4}$ cm/s ($3.3 \times 10^{-6}$ ft/s) | | | | | Assumed from Fig. 7.6 in Holtz and Kovacs for very fine sand, mixture of sand, silt and clay, stratified clay deposits |
| | $k_h/k_v$ | 4 | | | | | DWR (2008), page 3-4, Table 3-1 Permeability Values |
| | Undrained Shear Strength, $s_u$ | 1000 psf (CL-CH) | | | | X | Estimated from correlation of strength properties from SPT and CPT data |
| | Total Stress Cohesion, c | -- | | | | | |
| | Total Stress Friction Angle, $\phi$ | -- | | | | | |
| | Effective Cohesion, c' | 300 psf | X | X | X | | Table 5.8 in Duncan and Wright |
| | Effective Friction Angle, $\phi'$ | 30° | X | X | X | | Estimated from chart by Ladd (1977) using average PI=22, and Table 5.8 in Duncan and Wright |
| Upper Clay | Unit Weight, $\gamma_t$ | 125 pcf | | | | | Average of all borings in upper clay layer |
| | Permeability, $k_v$ | $1.0 \times 10^{-6}$ cm/s ($3.28 \times 10^{-8}$ ft/s) | | | | | Assumed from Fig. 7.6 in Holtz and Kovacs for stratified clay deposits |
| | $k_h/k_v$ | 10 | | | | | DWR (2008), page 3-4, Table 3-1 Permeability Values |
| | Undrained Shear Strength, $s_u$ | 1000 psf * | | | | X | Estimated from correlation of strength properties from SPT data and CPT data, * Reach A uses 1200 psf, Reach C and A1 uses varying strength with depth plot, Reach E uses 1400 psf |
| | Total Stress Cohesion, c | -- | | | | | |
| | Total Stress Friction Angle, $\phi$ | -- | | | | | |
| | Effective Cohesion, c' | 300 psf | X | X | X | | Table 5.8 in Duncan and Wright |
| | Effective Friction Angle, $\phi'$ | 30° | X | X | X | | Estimated from chart by Ladd (1977) using average PI=16, and Table 5.8 in Duncan and Wright |

**TABLE 1: MATERIAL PROPERTIES**

| Uvas Creek Soil Properties | | | | | | | |
|---|---|---|---|---|---|---|---|
| Soil Type | Properties | | Existing | Steady-state | Drawdown | Seismic | Source |
| Sand | Unit Weight, $\gamma_t$ | 140 pcf | | | | | Average of all borings in sand layer |
| | Permeability, $k_v$ | $8.52 \times 10^{-2}$ cm/s ($2.8 \times 10^{-3}$ ft/s) | | | | | Kozeny-Carman average value for sand layer |
| | $k_h/k_v$ | 4 | | | | | DWR (2008), page 3-4, Table 3-1 Permeability Values |
| | Undrained Shear Strength, $s_u$ | -- | | | | | |
| | Total Stress Cohesion, c | 0 psf | | | | X | |
| | Total Stress Friction Angle, $\phi$ | 32° | | | | X | Estimated from correlation of strength properties from blow counts - Meyerhof (35) and from CPT data (35) |
| | Effective Cohesion, c' | 0 psf | | | | | |
| | Effective Friction Angle, $\phi'$ | 32° | X | X | X | | Assume same as $\phi$ |
| Soft to Medium Stiff Clay (Cross Sections A1 & C only) | Unit Weight, $\gamma_t$ | 114 pcf | | | | | Average of all borings in soft to medium stiff clay layer |
| | Permeability, $k_v$ | $1.0 \times 10^{-6}$ cm/s ($3.28 \times 10^{-8}$ ft/s) | | | | | Assumed from Fig. 7.6 in Holtz and Kovacs for stratified clay deposits |
| | $k_h/k_v$ | 10 | | | | | DWR (2008), page 3-4, Table 3-1 Permeability Values |
| | Undrained Shear Strength, $s_u$ | varies see note | | | | X | Reach C = 350 psf, Reach A1 = 500 psf, Estimated from correlation of strength properties CPT data |
| | Total Stress Cohesion, c | -- | | | | | |
| | Total Stress Friction Angle, $\phi$ | -- | | | | | |
| | Effective Cohesion, c' | 100 psf | X | X | X | | Table 5.8 in Duncan and Wright |
| | Effective Friction Angle, $\phi'$ | 29° | X | X | X | | Estimated from chart by Ladd (1977) using average PI=20 |

**TABLE 1: MATERIAL PROPERTIES**

| Uvas Creek Soil Properties | | | | | | | |
|---|---|---|---|---|---|---|---|
| Soil Type | Properties | | Existing | Steady-state | Drawdown | Seismic | Source |
| Gravel | Unit Weight, $\gamma_t$ | 148 pcf | | | | | Average of all borings in gravel layer |
| | Permeability, $k_v$ | $2.36 \times 10^{-1}$ cm/s ($7.74 \times 10^{-3}$ ft/s) | | | | | Kozeny-Carman average value for gravel layer |
| | $k_h/k_v$ | 10 | | | | | DWR (2008), page 3-4, Table 3-1 Permeability Values |
| | Undrained Shear Strength, $s_u$ | -- | | | | | Estimated from correlation of strength properties from SPT data and CPT data |
| | Total Stress Cohesion, c | 0 psf | | | | X | |
| | Total Stress Friction Angle, $\phi$ | 34° | | | | X | Assumed same as effective strength |
| | Effective Cohesion, c' | 0 psf | X | X | X | | |
| | Effective Friction Angle, $\phi'$ | 34° | X | X | X | | Estimated from SPT and CPT data |
| Lower Clay | Unit Weight, $\gamma_t$ | 135 pcf | | | | | Based on B-3 clay sample at 70 feet |
| | Permeability, $k_v$ | $1.0 \times 10^{-6}$ cm/s ($3.28 \times 10^{-8}$ ft/s) | | | | | Assumed from Fig. 7.6 in Holtz and Kovacs for stratified clay deposits |
| | $k_h/k_v$ | 10 | | | | | DWR (2008), page 3-4, Table 3-1 Permeability Values |
| | Undrained Shear Strength, $s_u$ | 1700 psf ** | | | | X | Estimated from correlation of strength properties from SPT and CPT data, ** Reach B uses 1500 psf |
| | Total Stress Cohesion, c | -- | | | | | |
| | Total Stress Friction Angle, $\phi$ | -- | | | | | |
| | Effective Cohesion, c' | 300 psf | X | X | X | | Table 5.8 in Duncan and Wright |
| | Effective Friction Angle, $\phi'$ | 29° | X | X | X | | Estimated from chart by Ladd (1977) using average PI=20 |

**TABLE 1: MATERIAL PROPERTIES**

| Uvas Creek Soil Properties | | | | | | | |
|---|---|---|---|---|---|---|---|
| Soil Type | Properties | | Existing | Steady-state | Drawdown | Seismic | Source |
| Rip Rap | Unit Weight, $\gamma_t$ | 145 pcf | | | | | Assumed |
| | Permeability, $k_v$ | 1 cm/s (0.0328 ft/s) | | | | | Assumed from Fig. 7.6 in Holtz and Kovacs for good drainage material |
| | $k_h/k_v$ | 10 | | | | | DWR (2008), page 3-4, Table 3-1 Permeability Values |
| | Undrained Shear Strength, $s_u$ | -- | | | | | Estimated from correlation of strength properties from SPT data and CPT data |
| | Total Stress Cohesion, c | 0 psf | | | | X | |
| | Total Stress Friction Angle, $\phi$ | 45° | | | | X | Assumed rock |
| | Effective Cohesion, c' | 0 psf | X | X | X | | |
| | Effective Friction Angle, $\phi'$ | 45° | X | X | X | | Assumed rock |

## TABLE 2: LEVEE SLOPE STABILITY ANALYSIS RESULTS

| Analysis Condition | Strength Type | Required minimum Factor of Safety | Computed Factor of Safety | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Reach A1 | | Reach A | | Reach B | | Reach C | | Reach D | | Reach E | |
| | | | W | L | W | L | W | L | W | L | W | L | W | L |
| Existing | Drained | 1.3 | 2.16 | 3.69 | 2.40 | 4.66 | 5.06 | 4.87 | 1.99 | 4.48 | 4.74 | 4.54 | 2.06 | 5.87 |
| Steady-State Seepage | Drained | 1.4[1] | -- | 3.23 | -- | 4.07 | -- | 4.31 | -- | 3.85 | -- | 3.58 | -- | 4.61 |
| Rapid Drawdown | 3-stage | 1.2[1] | 1.50 | -- | 1.68 | -- | 3.54 | -- | 1.34 | -- | 3.18 | -- | 1.30 | -- |
| Pseudo-Static Screening | Undrained | 1.15 | 1.14 | 3.05 | 1.36 | 2.46 | 1.72 | 1.55 | 1.13 | 2.28 | 3.10 | 3.32 | 1.41 | 2.71 |
| **Seismic Yield Coefficient** | | | | | | | | | | | | | | |
| Seismic Yield Acceleration | Undrained | n/a | 1.00 | 1.00 | 1.00 | 1.00 | 0.99 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| | | ky (g) | 0.193 | 0.538 | 0.29 | 0.423 | 0.29 | 0.273 | 0.193 | 0.377 | 0.445 | 0.60 | 0.258 | 0.45 |
| **Maximum Exit Gradient** | | | | | | | | | | | | | | |
| Maximum Exit Gradient | Steady-State Seepage | | | 0.35 | | 0.24 | | 0.35 | | 0.12 | | 0.20 | | 0.25 |
| **Range of Seismic Deformation[2] (inches)** | | | | | | | | | | | | | | |
| Seismic Deformation | | | 17-49 | 0-0 | 8-25 | 1-4 | 8-25 | 11-30 | 14-38 | 2-7 | 1-3 | 0-0 | 14-38 | 1-2 |
| **Range of Seismically Induced Settlements[3] (inches)** | | | | | | | | | | | | | | |
| Seismic Settlement | | | 1.0-3.0 | | 1.0-3.0 | | N/A | | 0.5-1.0 | | 1.5-3.0 | | 0.5-1.0 | |

Notes:

[1] USACE, 2000

[2] Makdisi and Seed (1978) & Seed (1979)

[3] CLiq analysis (GeoLogismiki 2006)

W denotes waterside. L denotes landside.

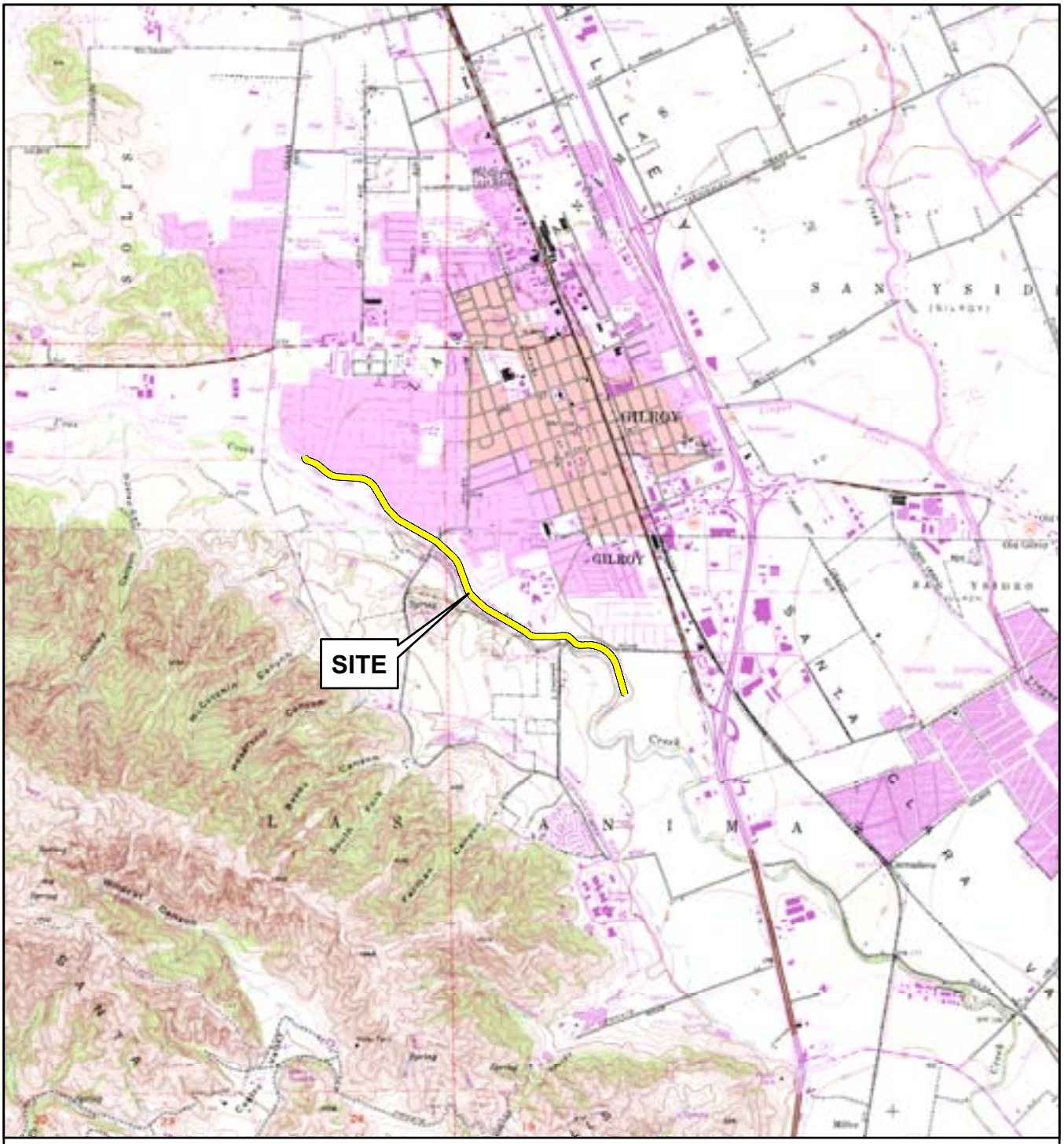
## TABLE 3: LOMA PRIETA SEISMIC DEFORMATION ANALYSIS

| Section | Side | $k_y$ Determination | | Deformations from Makdisi Seed | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $k_y$ | $k_y/k_{0.43}$ | M = 6 1/2 High (cm) | M = 6 1/2 Low (cm) | M = 7 1/2 High (cm) | M = 7 1/2 Low (cm) | Low (cm) | High (cm) | Low (in) | High (in) |
| A1 | Water | 0.193 | 0.55 | 10 | 1.2 | 10.7 | 2.2 | 1.70 | 10.35 | 0.7 | 4.1 |
| A1 | Land | 0.538 | 1.25 | 0.0 | 0.0 | 0.0 | 0.0 | 0.00 | 0.00 | 0.0 | 0.0 |
| A | Water | 0.29 | 0.67 | 4.6 | 0.46 | 6.6 | 0.84 | 0.65 | 5.60 | 0.3 | 2.2 |
| A | Land | 0.423 | 0.98 | 0.0 | 0.0 | 0.0 | 0.0 | 0.00 | 0.00 | 0.0 | 0.0 |
| B | Water | 0.29 | 0.67 | 4.6 | 0.46 | 6.6 | 0.84 | 0.65 | 5.60 | 0.3 | 2.2 |
| B | Land | 0.273 | 0.63 | 5.9 | 0.63 | 9.2 | 1.1 | 0.87 | 7.55 | 0.3 | 3.0 |
| C | Water | 0.193 | 0.60 | 7.4 | 0.82 | 12 | 1.45 | 1.14 | 9.70 | 0.4 | 3.8 |
| C | Land | 0.377 | 0.88 | 0.0 | 0.0 | 0.0 | 0.0 | 0.00 | 0.00 | 0.0 | 0.0 |
| D | Water | 0.445 | 1.03 | 0.0 | 0.0 | 0.0 | 0.0 | 0.00 | 0.00 | 0.0 | 0.0 |
| D | Land | 0.60 | 1.40 | 0.0 | 0.0 | 0.0 | 0.0 | 0.00 | 0.00 | 0.0 | 0.0 |
| E | Water | 0.258 | 0.60 | 7.4 | 0.82 | 12 | 1.45 | 1.14 | 9.70 | 0.4 | 3.8 |
| E | Land | 0.45 | 1.05 | 0.0 | 0.0 | 0.0 | 0.0 | 0.00 | 0.00 | 0.0 | 0.0 |

Section C waterside seismic reduction factor x= 15.85
(Youd 2001) rd= 0.75

Section A1 waterside seismic reduction factor x= 13.41
(Youd 2001) rd= 0.82

# FIGURES




Base map from USGS 7.5' Chittenden and Gilroy, California topographic quadrangles.

N

0 2,000 4,000 Feet

SITE LOCATION MAP
Uvas Creek Levee
FEMA Levee Certification
Gilroy, California

| By: DMO | Date: 4/10/2009 | Project No. 13815.001 |
|---|---|---|
| AMEC Geomatrix | | Figure 1 |

File path: S:\13800\13815\13815.001\task_a3\09_0409_uclr_fig_01.mxd

Explanation

- CPT, Pacific Geotechnical Engineering, 2008, this study
- Boring, Pacific Geotechnical Engineering, 2008, this study
- Boring, Woodward-Clyde Consultants, August 1987, for Ruth and Going Inc.
- Boring, Anderson Geotechnical, August 1983, for U.S. Army Corps of Engineers
- Boring, J.F. Kleinfelder and Associates, July 1981, for U.S. Army Corps of Engineers

0 100 200 Feet

N

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP, SOUTHERN REACH
Uvas Creek Levee
FEMA Levee Certification
Gilroy, California

| By: DMO | Date: 4/10/2009 | Project No. 13815.001 |
|---|---|---|
| AMEC Geomatrix | | Figure 2 |

File path: S:\13800\13815\13815.001\task_a3\09_0409_uclr_fig_05_rotated_fig_2.mxd

Explanation

- CPT, Pacific Geotechnical Engineering, 2008, this study
- Boring, Pacific Geotechnical Engineering, 2008, this study
- Boring, Woodward-Clyde Consultants, August 1987, for Ruth and Going Inc.
- Boring, Anderson Geotechnical, August 1983, for U.S. Army Corps of Engineers
- Boring, J.F. Kleinfelder and Associates, July 1981, for U.S. Army Corps of Engineers

0 100 200 Feet

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP, SOUTH CENTRAL REACH
Uvas Creek Levee
FEMA Levee Certification
Gilroy, California

| By: DMO | Date: 4/10/2009 | Project No. 13815.001 |
|---|---|---|
| AMEC Geomatrix | | Figure 3 |

File path: S:\13800\13815\13815.001\task_a3\09_0409_uclr_fig_04_rotated_fig_3.mxd

Explanation

- CPT, Pacific Geotechnical Engineering, 2008, this study
- Boring, Pacific Geotechnical Engineering, 2008, this study
- Boring, Woodward-Clyde Consultants, August 1987, for Ruth and Going Inc.
- Boring, Anderson Geotechnical, August 1983, for U.S. Army Corps of Engineers
- Boring, J.F. Kleinfelder and Associates, July 1981, for U.S. Army Corps of Engineers

0 100 200 Feet

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP, NORTH CENTRAL REACH
Uvas Creek Levee
FEMA Levee Certification
Gilroy, California

| By: DMO | Date: 4/10/2009 | Project No. 13815.001 |
|---|---|---|
| AMEC Geomatrix | | Figure 4 |

File path: S:\13800\13815\13815.001\task_a3\09_0409_uclr_fig_03_rotated_fig_4.mxd

Explanation

- CPT, Pacific Geotechnical Engineering, 2008, this study
- Boring, Pacific Geotechnical Engineering, 2008, this study
- Boring, Woodward-Clyde Consultants, August 1987, for Ruth and Going Inc.
- Boring, Anderson Geotechnical, August 1983, for U.S. Army Corps of Engineers
- Boring, J.F. Kleinfelder and Associates, July 1981, for U.S. Army Corps of Engineers

0 100 200 Feet

N

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP, NORTHERN REACH
Uvas Creek Levee
FEMA Levee Certification
Gilroy, California

| By: DMO | Date: 4/10/2009 | Project No. 13815.001 |
|---|---|---|
| AMEC Geomatrix | | Figure 5 |

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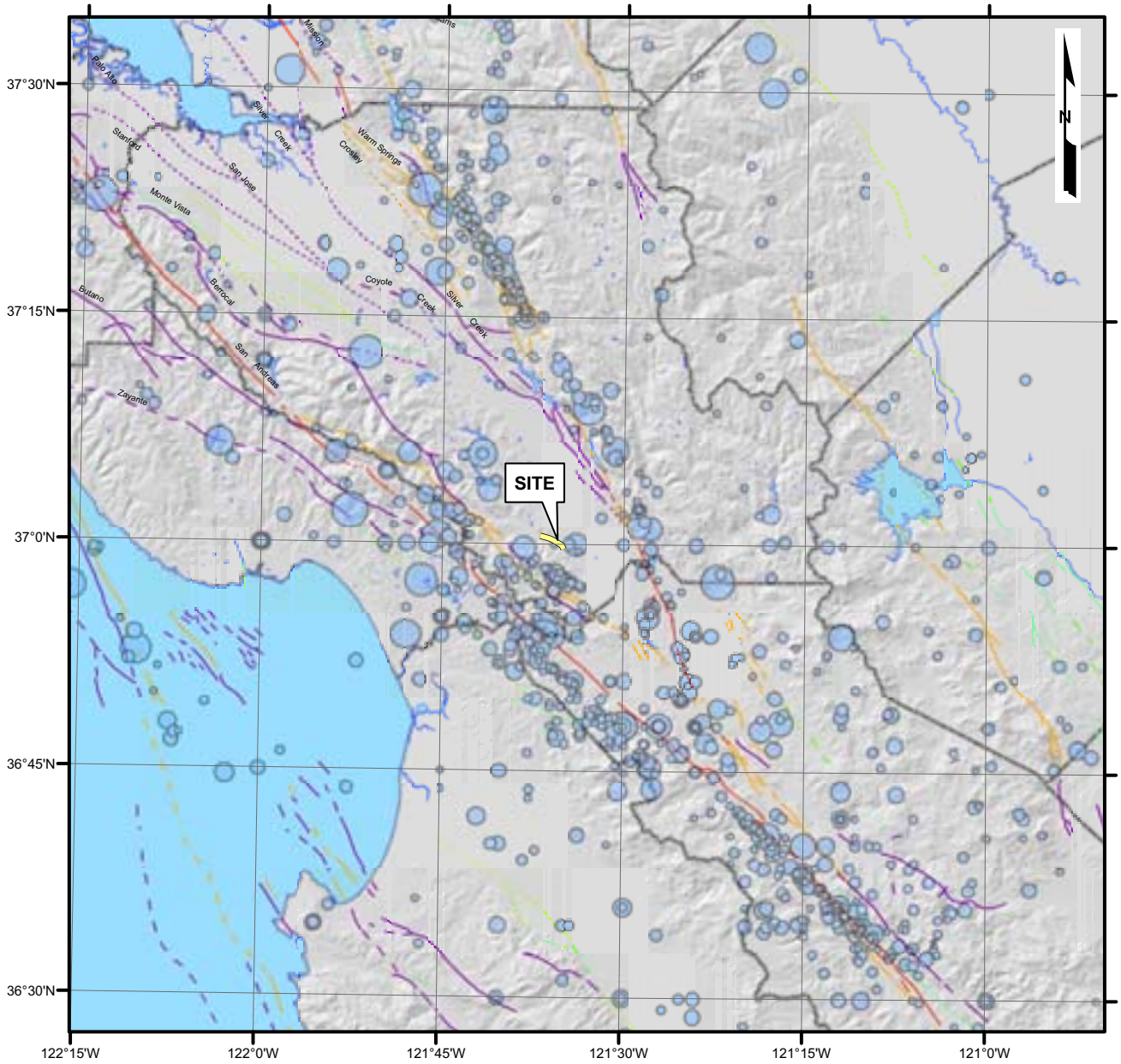


## Explanation

Fault traces on land shown as solid where well located, dashed where uncertain or inferred, dotted where buried. Data from Jennings (1994).

- Faults with historic surface rupture/creep
- Faults that displace Holocene (~11 ka) or latest Pleistocene (~20 Ka) deposits or geomorphic surfaces.
- Faults that displace Late Quaternary (~ 780 ka) deposits or geomorphic surfaces.
- Quaternary faults, undifferentiated (1.8 Ma).

Independant Seismicity (1769-2008)

- 3.0
- 4.0
- 5.0
- 6.0
- 7.0

0 5 10 Miles

REGIONAL FAULT AND HISTORIC SEISMICITY MAP
Uvas Creek Levee
FEMA Levee Certification
Gilroy, California

| By: DAL | Date: 4/9/2009 | Project No. 13815.001 |
|---|---|---|
| AMEC Geomatrix | | Figure 6 |

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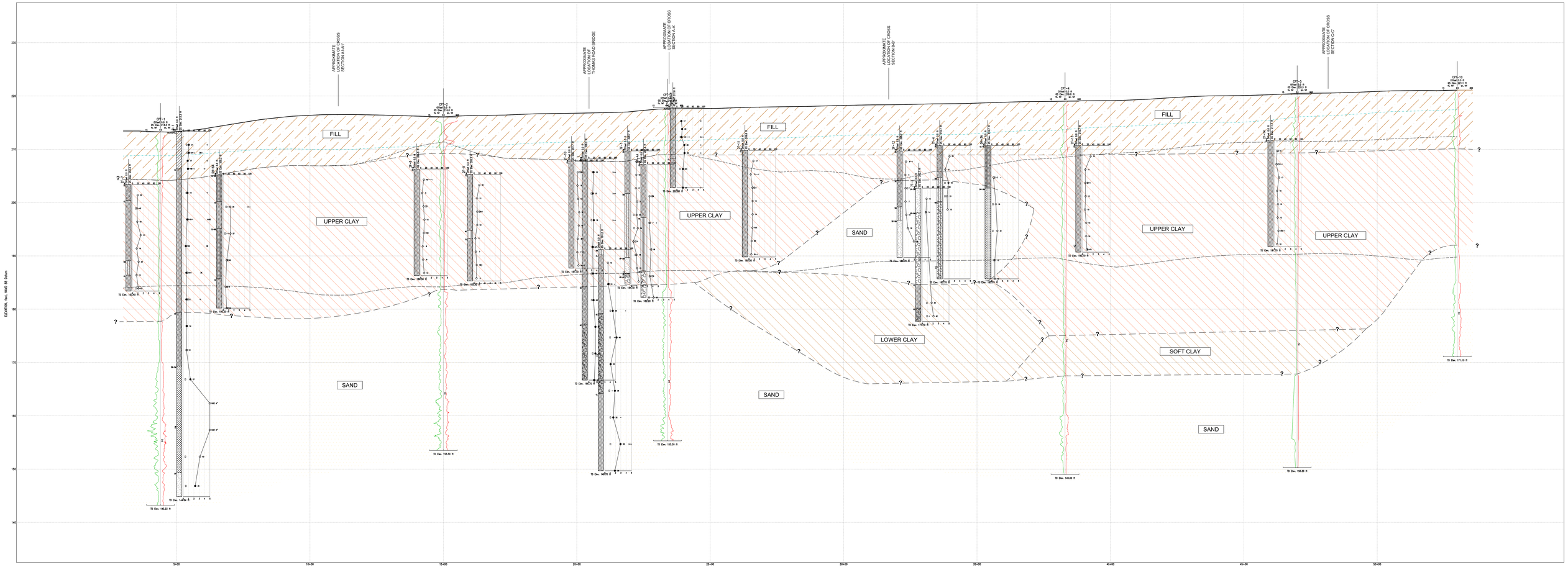



LEVEE STATIONING, feet

**Surfaces Legend**

- CREST ELEVATION
- WATER SURFACE ELEVATION (PROJECTED FROM CHANNEL)
- NATURAL GROUND ELEVATION (PROJECTED FROM LANDSIDE TOE)
- CHANNEL INVERT ELEVATION (PROJECTED FROM CHANNEL)

**Boring Plot Legend**

Blow Ind Soil
- SPT
- Bulk Sample (Modified, 0.42)

Water Content, %

Pl, Ll, %

Sw,Sw2
- Pocket Pen
- Torvane

Water Level at Time of Drilling

Water Level at End of Drilling

Vertical Scale = 5.00 ft/in.
Horizontal Scale = 100.00 ft/in.

Begin Line North 100.00 East 0.00
End Line North 100.00 East 1140.00

**LIST OF BORING NUMBER AND OFFSET**

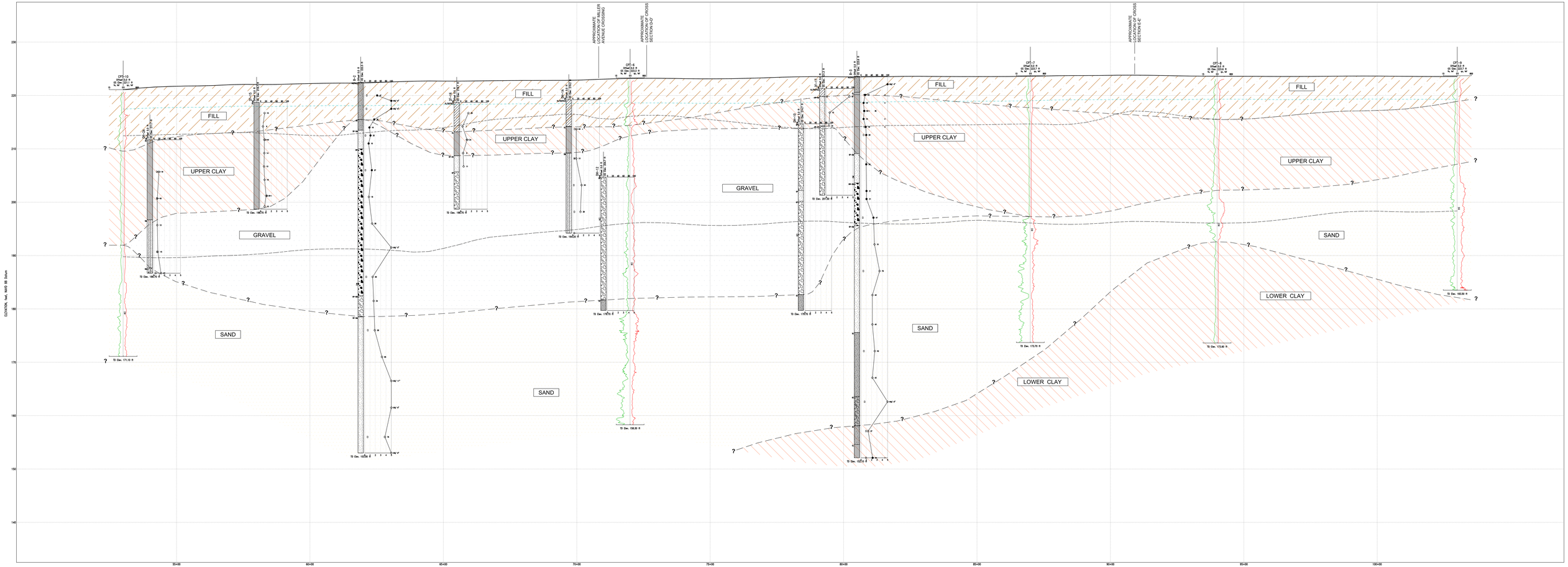
| | | |
|---|---|---|
| CPT-1 | -0.5 in. | (-50.0 ft) |
| B1-4 | 0.5 in. | (50.0 ft) |
| B-3 | 0.5 in. | (50.0 ft) |

**NOTES**

1. PROFILE ELEVATIONS WERE PROVIDED BY SCHAAFF & WHEELER.
2. ELEVATIONS ARE APPROXIMATE.
3. BORING AND CPT LOCATIONS ARE SHOWN IN FIGURES 2 THROUGH 5. ELEVATIONS OF BORINGS AND CPTS WERE ESTIMATED BASED ON PROFILE ELEVATIONS.
4. GEOLOGIC INTERPRETATIONS ARE GENERALIZED AND CONTACT ELEVATIONS ARE APPROXIMATE AND BASED ON AVAILABLE INFORMATION.

SECTION: Levee Profile 1/2
Uvas Creek Levee
FEMA Levee Certification
Gilroy, CA

| Rev | JH | Date: 02/27/09 | Project: 13615.001 |
|---|---|---|---|
| | | | Figure 7 |




LEVEE STATIONING, feet

**Surfaces Legend**

- CREST ELEVATION
- WATER SURFACE ELEVATION (PROJECTED FROM CHANNEL)
- NATURAL GROUND ELEVATION (PROJECTED FROM LANDSIDE TOE)
- CHANNEL INVERT ELEVATION (PROJECTED FROM CHANNEL)

**Boring Plot Legend**

Water Content
- Water Content, %
- P.I., %

Soil Symbol
- Plastic Fine
- Coarse

Water Level at Time of Drilling

Water Level at End of Drilling

Vertical Scale = 5.00 ft/in.
Horizontal Scale = 100.00 ft/in.

Begin Line North 100.00 East 0.00
End Line North 100.00 East 11400.00

**LIST OF BORING NUMBER AND OFFSET**

| | | |
|---|---|---|
| CPT-1 | -0.5 in. | (-50.0 ft) |
| B1-4 | 0.5 in. | (50.0 ft) |
| B-3 | 0.5 in. | (50.0 ft) |

**NOTES**

1. PROFILE ELEVATIONS WERE PROVIDED BY SCHAAFF & WHEELER.
2. ELEVATIONS ARE APPROXIMATE.
3. BORING AND CPT LOCATIONS ARE SHOWN IN FIGURES 2 THROUGH 5. ELEVATIONS OF BORINGS AND CPTS WERE ESTIMATED BASED ON PROFILE ELEVATIONS.
4. GEOLOGIC INTERPRETATIONS ARE GENERALIZED AND CONTACT ELEVATIONS ARE APPROXIMATE AND BASED ON AVAILABLE INFORMATION.

SECTION: Levee Profile 2/2
Uvas Creek Levee
FEMA Levee Certification
Gilroy, CA

| Rev | JH | Date: 02/27/09 | Project: 13815.001 |
|---|---|---|---|

Figure 8

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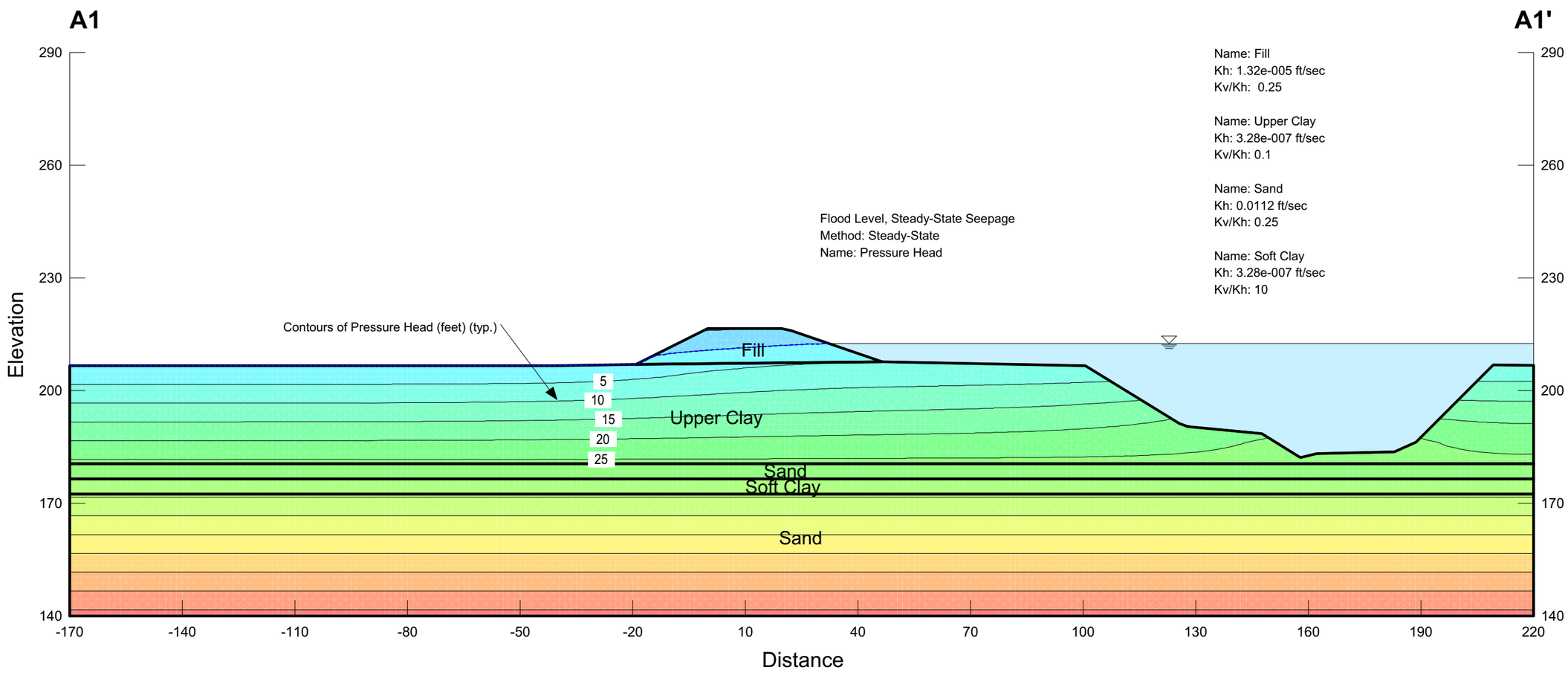


| | | | | | | |
|---|---|---|---|---|---|---|
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Steady-State Seepage Result**<br>Max. Flood Level<br>Cross Section A1<br>Levee Station 11+06 | **Figure**<br>9-1 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>6/18/2009 | Checked | | | |

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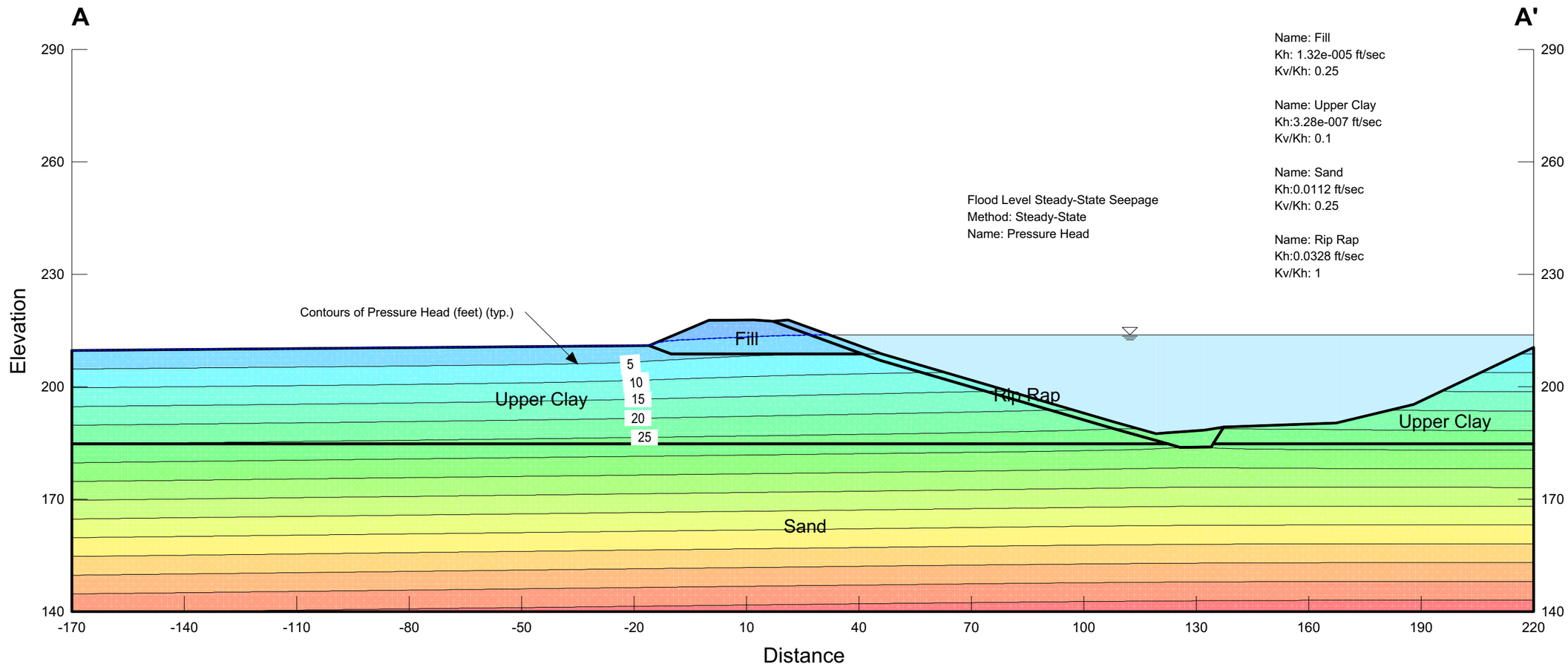


| | | | | | | |
|---|---|---|---|---|---|---|
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seepage Analysis Result**<br>Max. Flood Level<br>Cross Section A<br>Levee Station 23+46 | **Figure**<br>9-2 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>6/19/2009 | Checked | | | |

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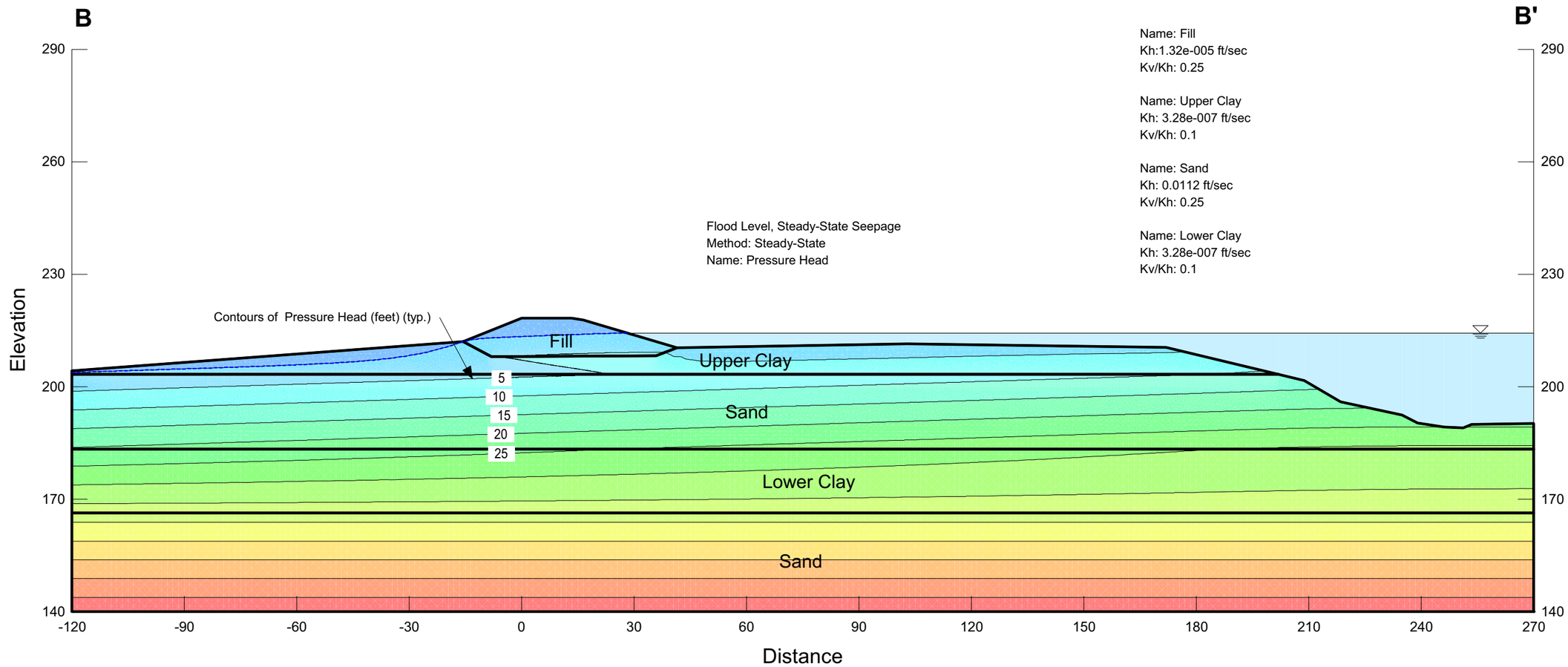
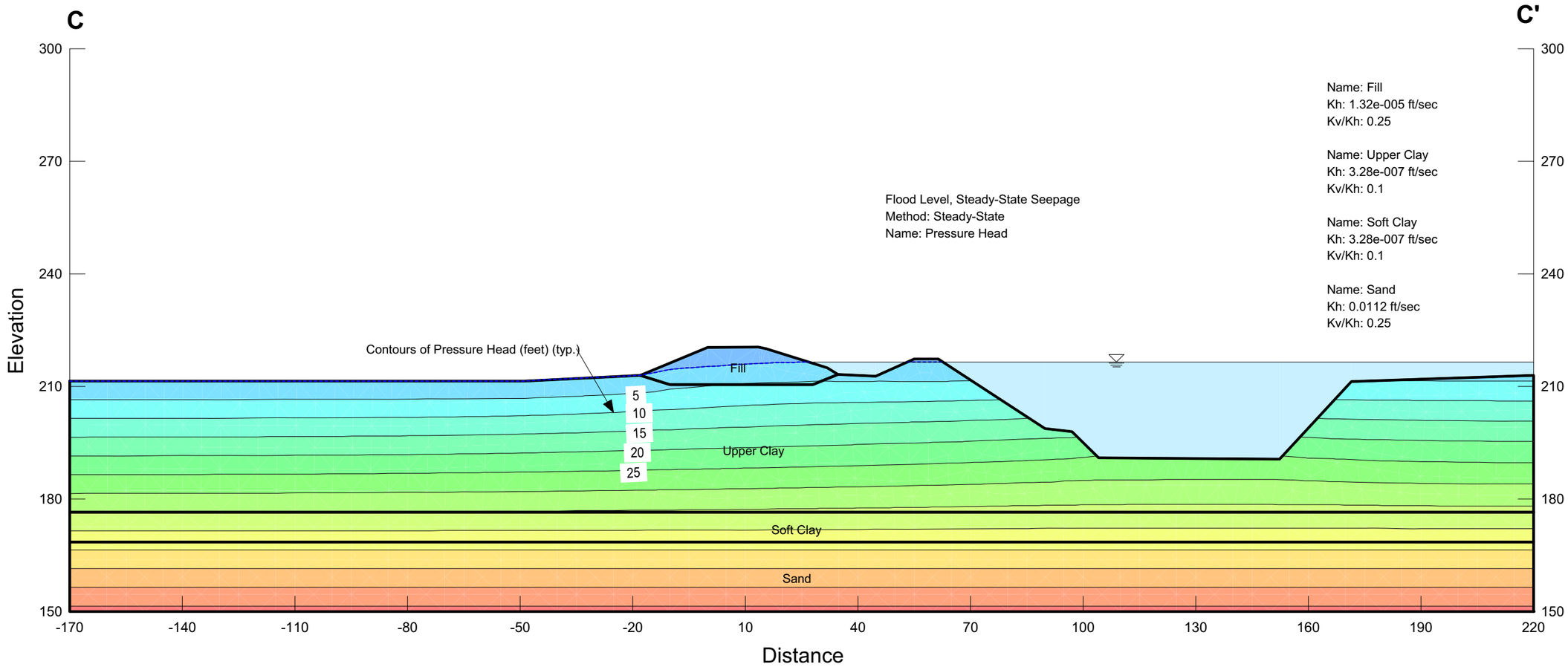



| | | | | | | |
|---|---|---|---|---|---|---|
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seepage Analysis Result**<br>Max. Flood Level<br>Cross Section B<br>Levee Station 31+69 | **Figure**<br>9-3 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>6/18/2009 | Checked | | | |




| | | | | | | |
|---|---|---|---|---|---|---|
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seepage Analysis Result**<br>Max. Flood Level<br>Cross Section C<br>Levee Station 48+16 | **Figure**<br>9-4 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>6/18/2009 | Checked | | | |

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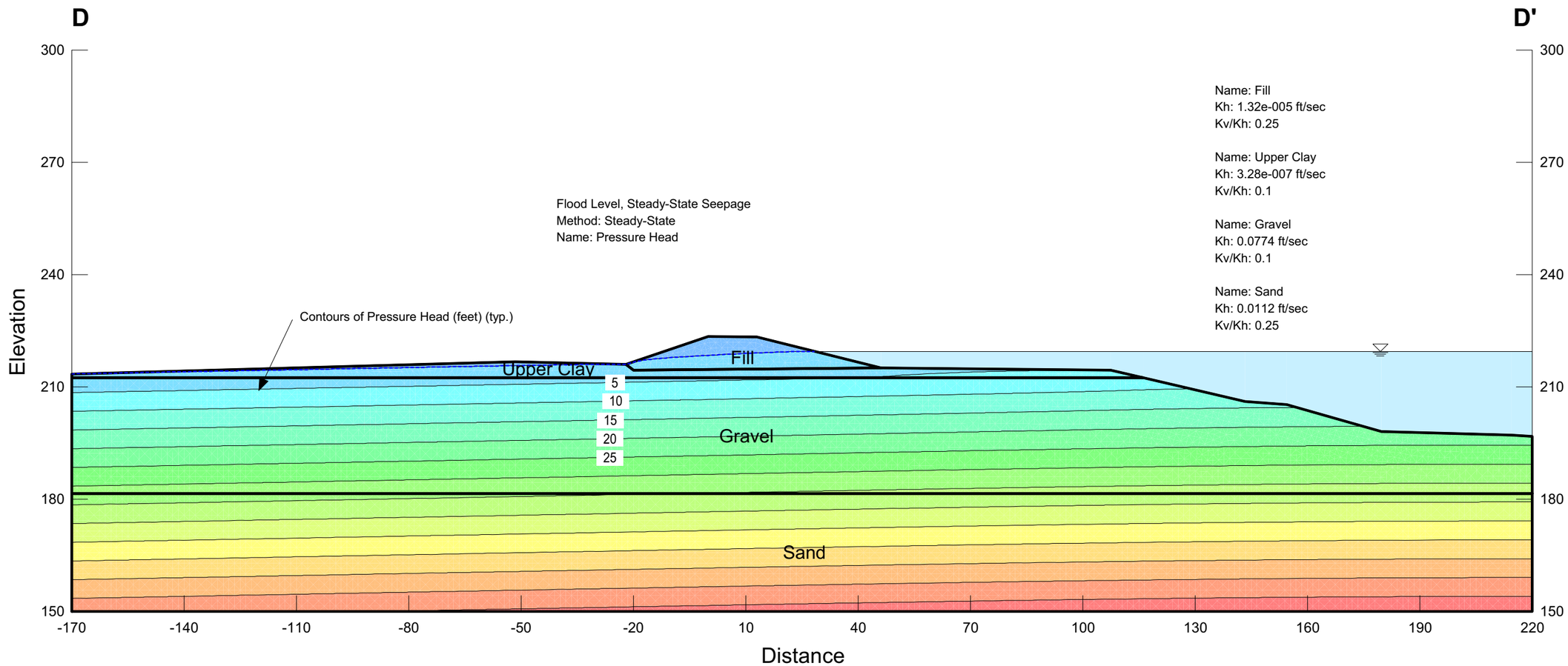


| | | | | | | |
|---|---|---|---|---|---|---|
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seepage Analysis Result**<br>Max. Flood Level<br>Cross Section D<br>Levee Station 72+62 | **Figure**<br>9-5 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>6/18/2009 | Checked | | | |

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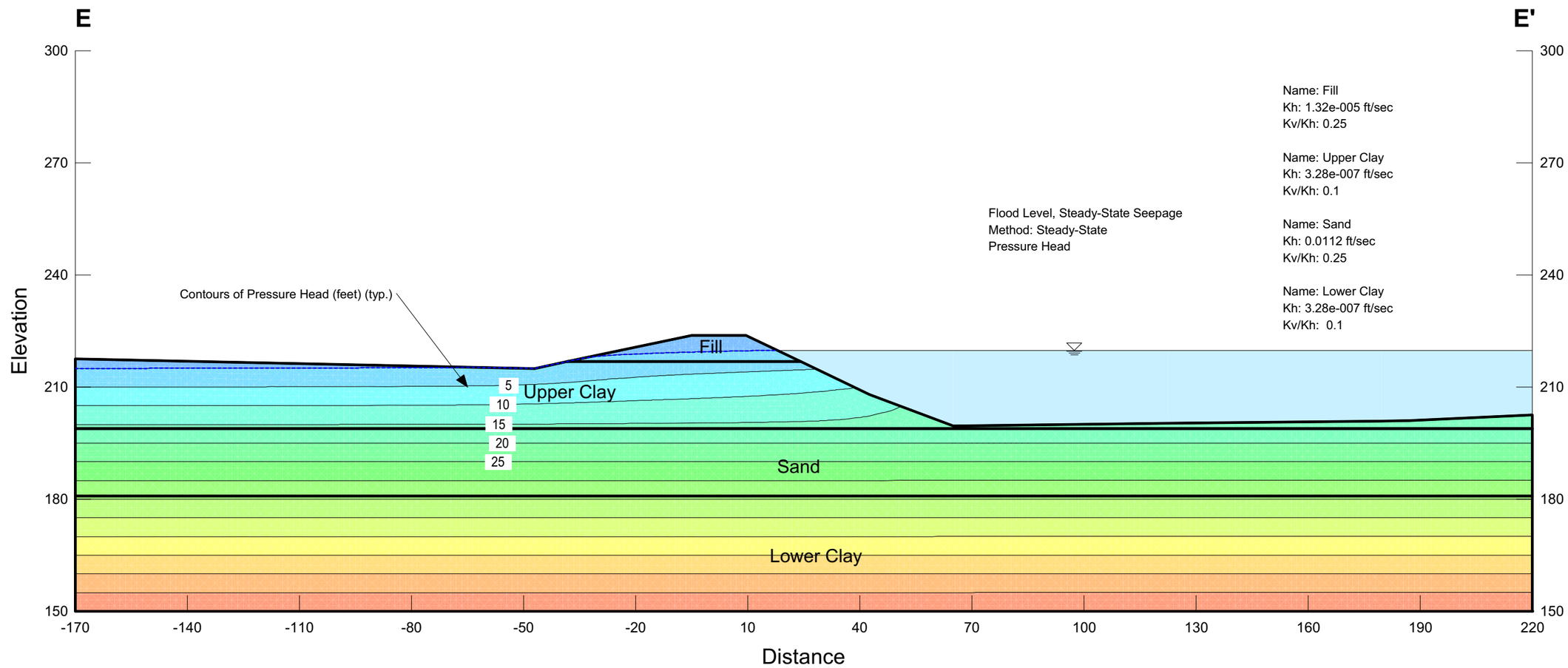


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seepage Analysis Result**<br>Max. Flood Level<br>Cross Section E<br>Levee Station 90+91 | **Figure**<br>9-6 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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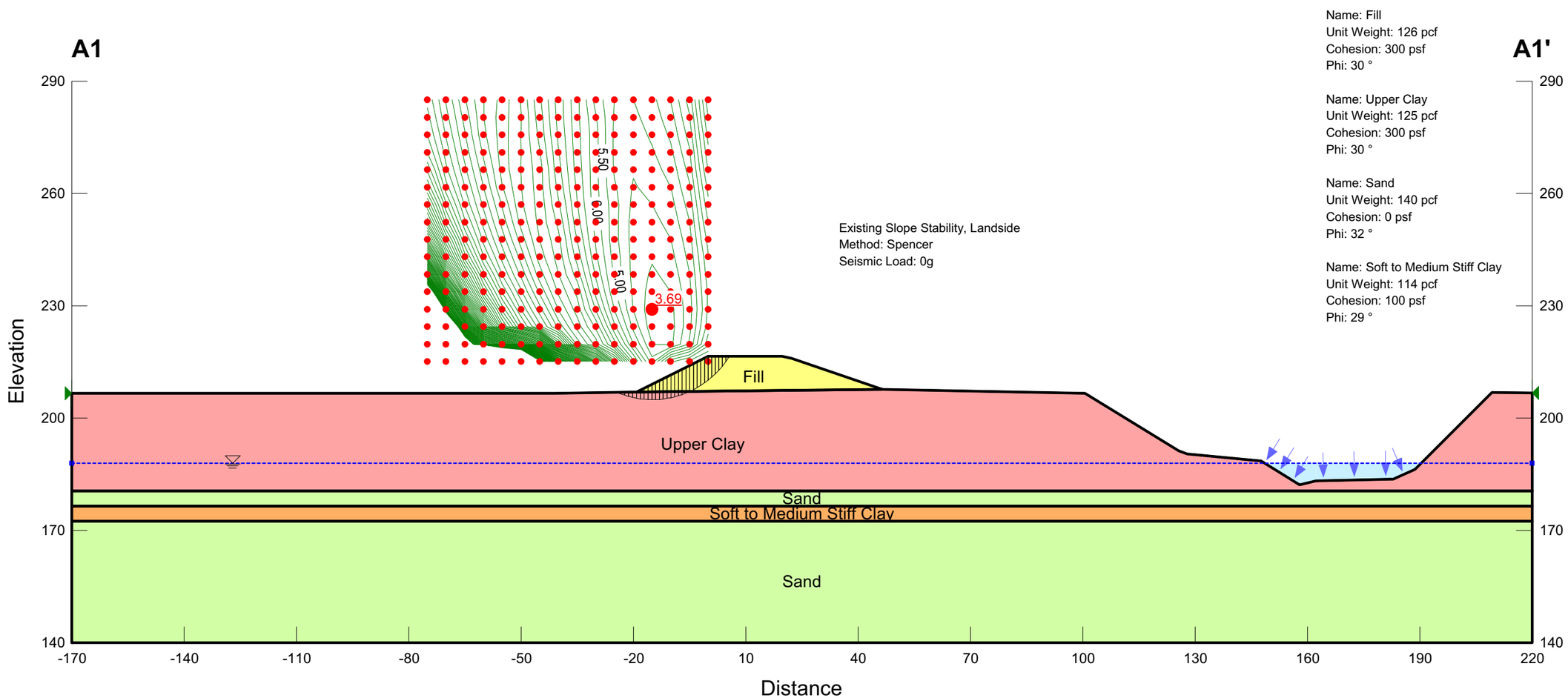


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Existing Slope Stability Result**<br>Landside<br>Cross Section A1<br>Levee Station 11+06 | **Figure**<br>10-1 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

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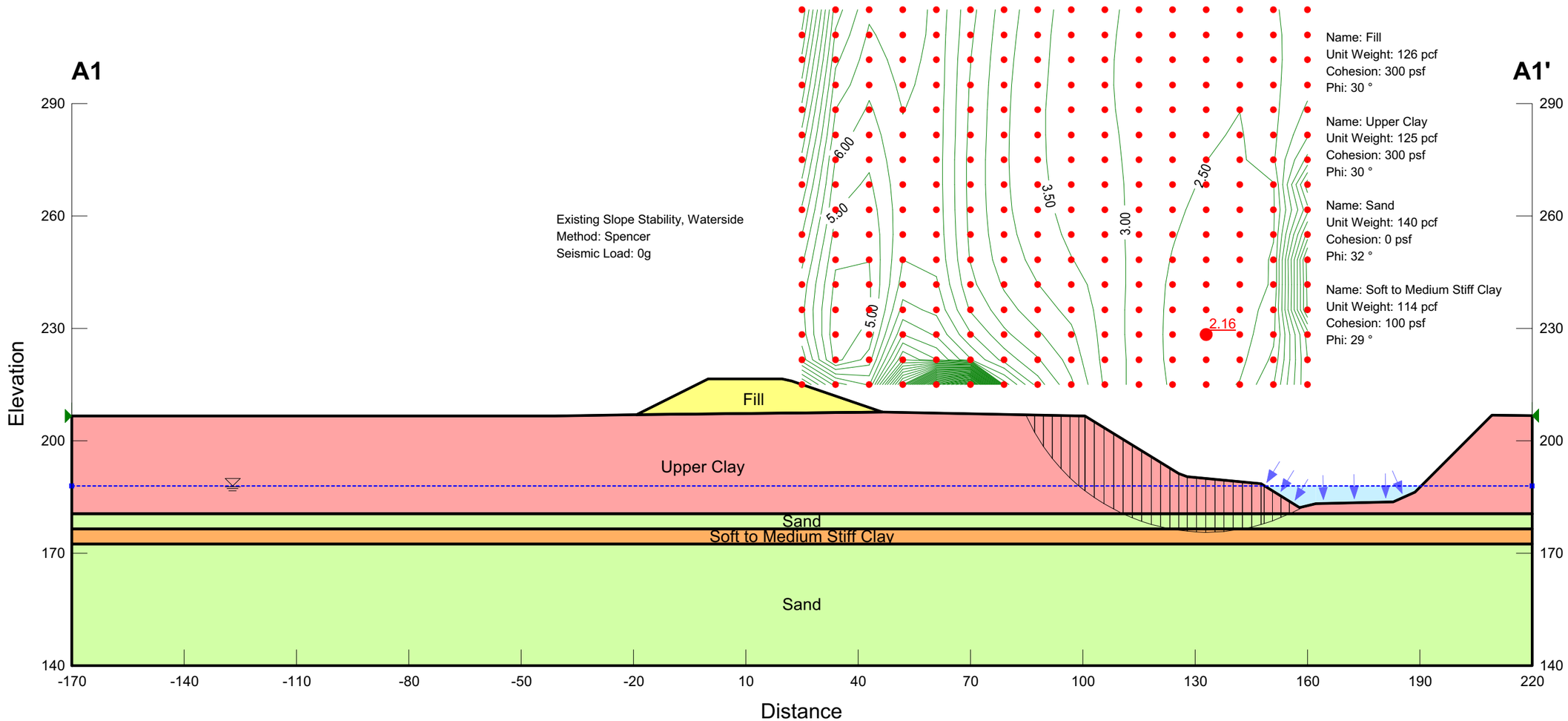


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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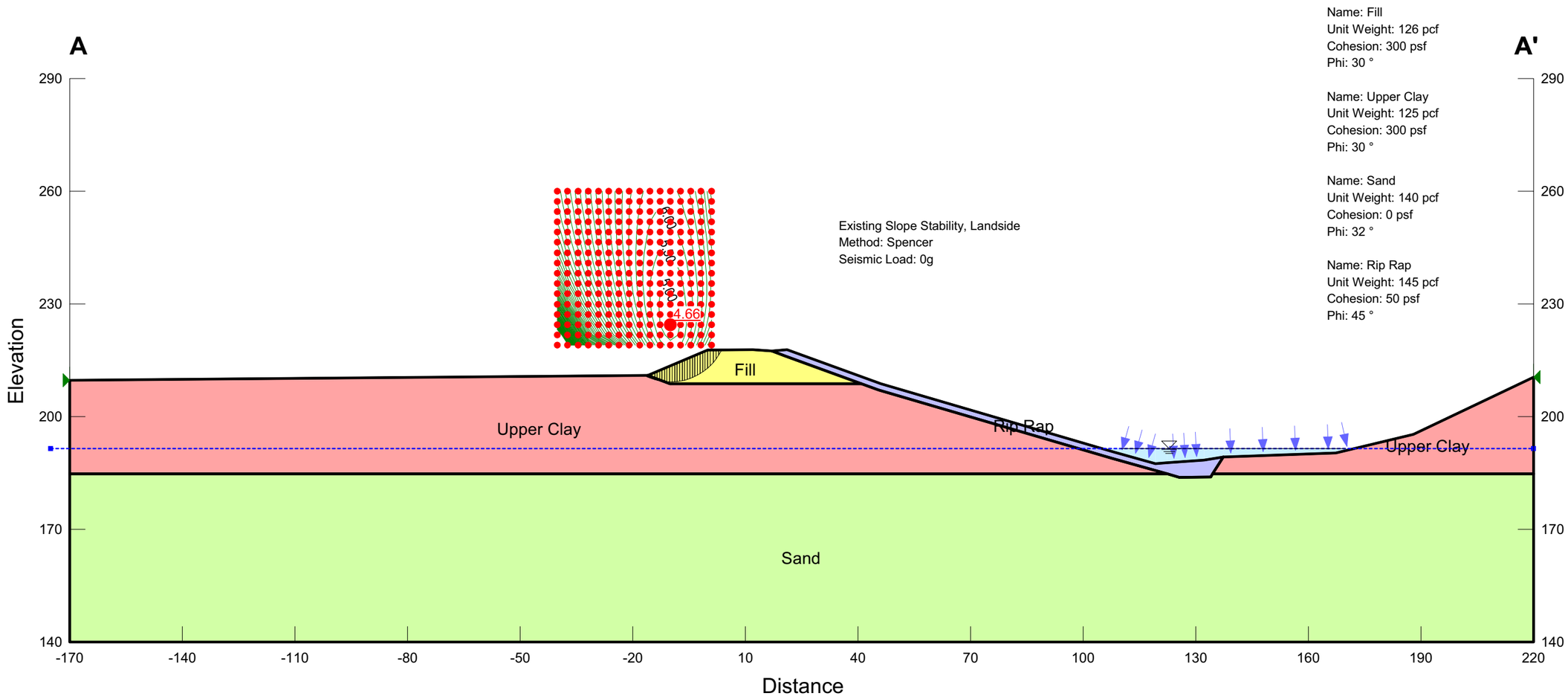


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/28/2009 | Checked | | | |

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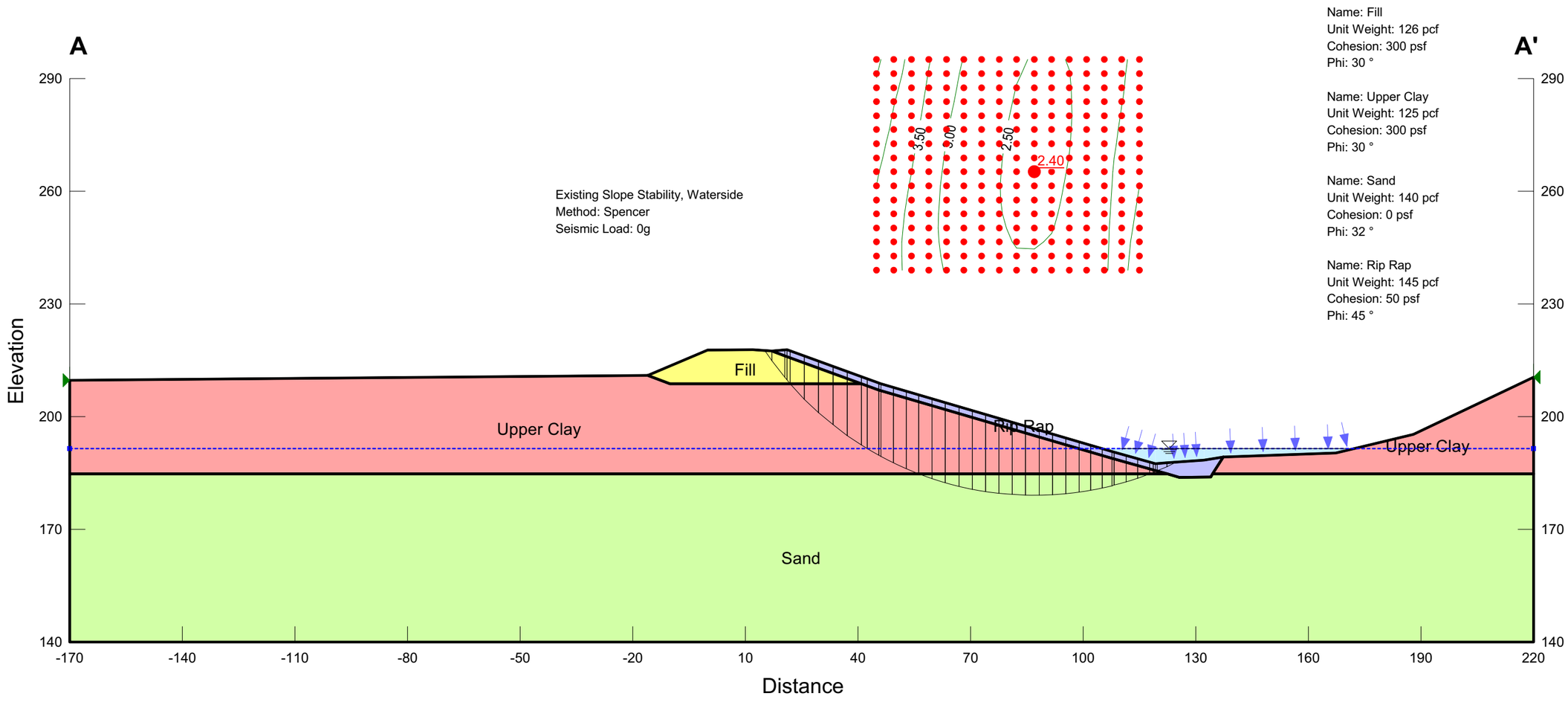


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/28/2009 | Checked | | | |

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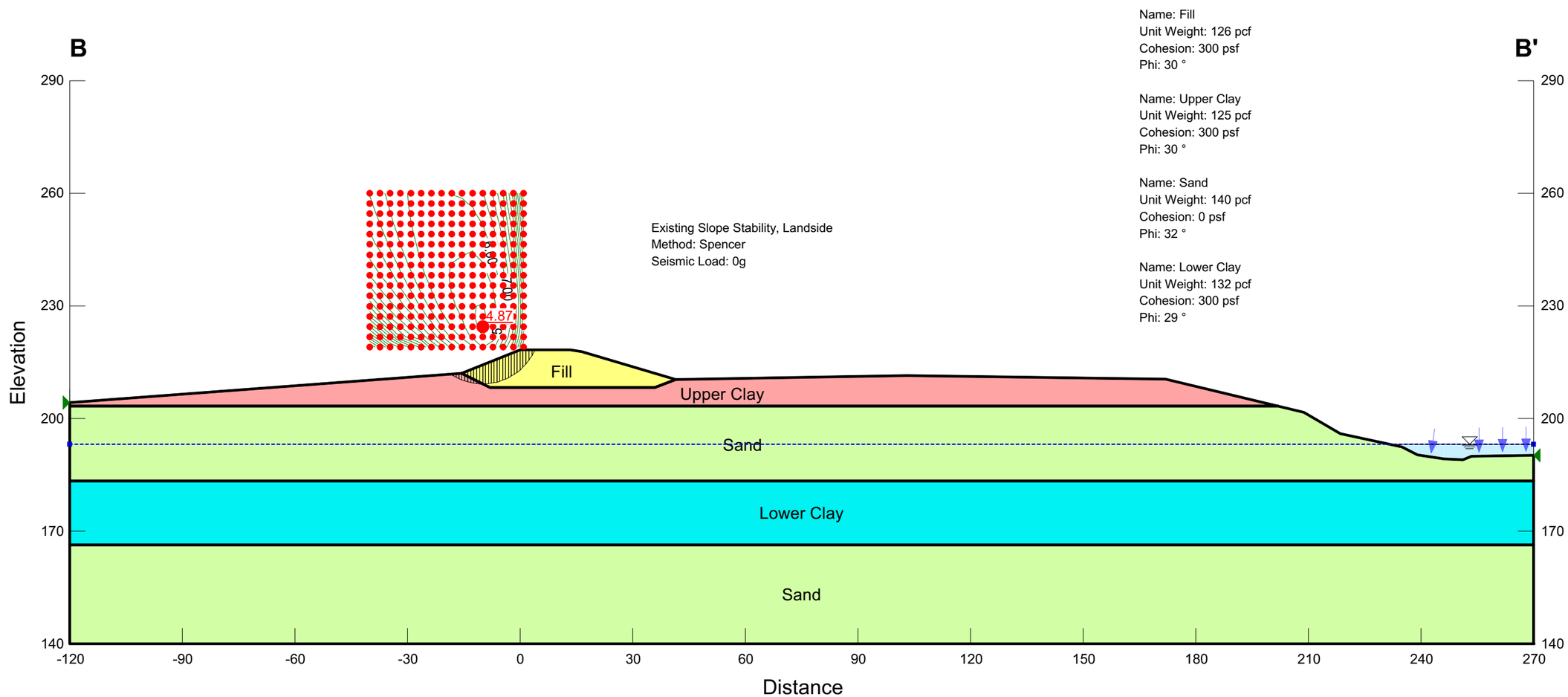


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/28/2009 | Checked | | | |

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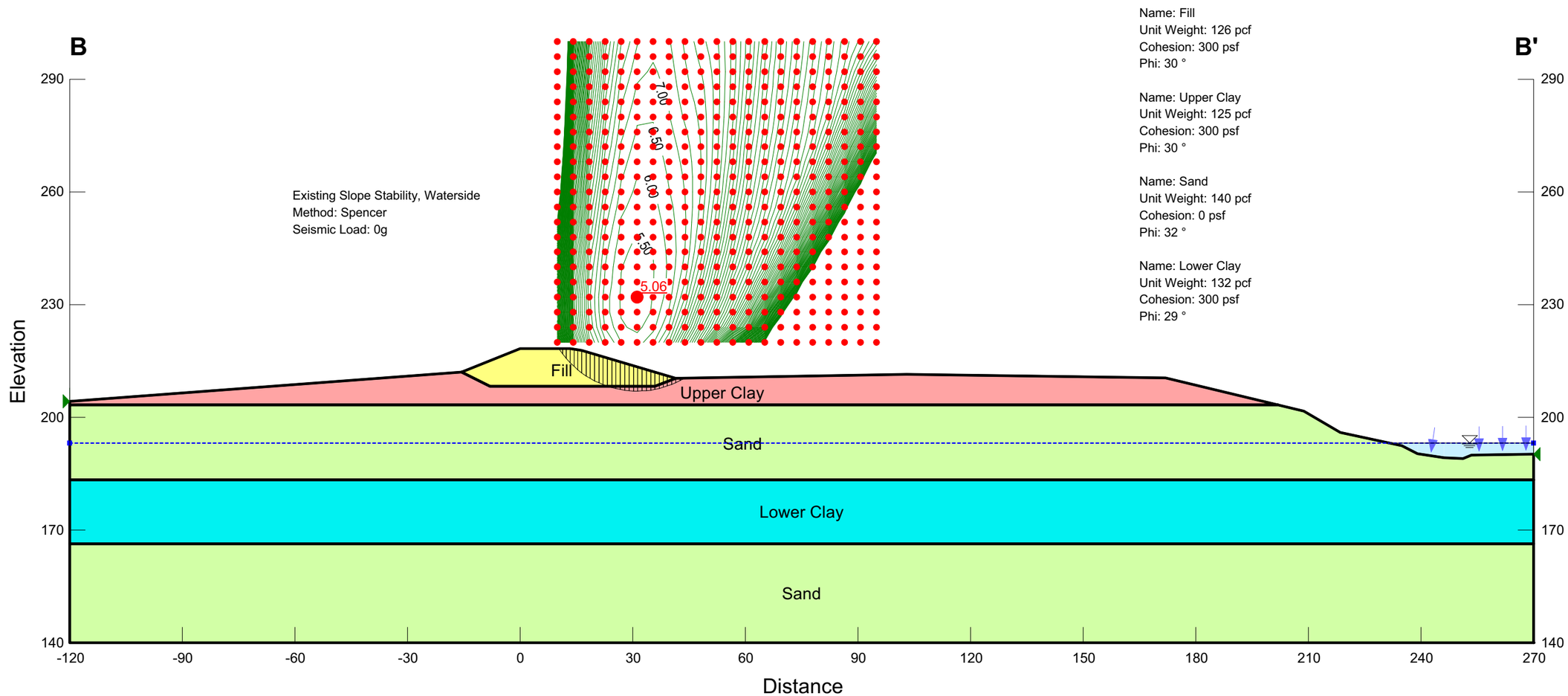


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/28/2009 | Checked | | | |

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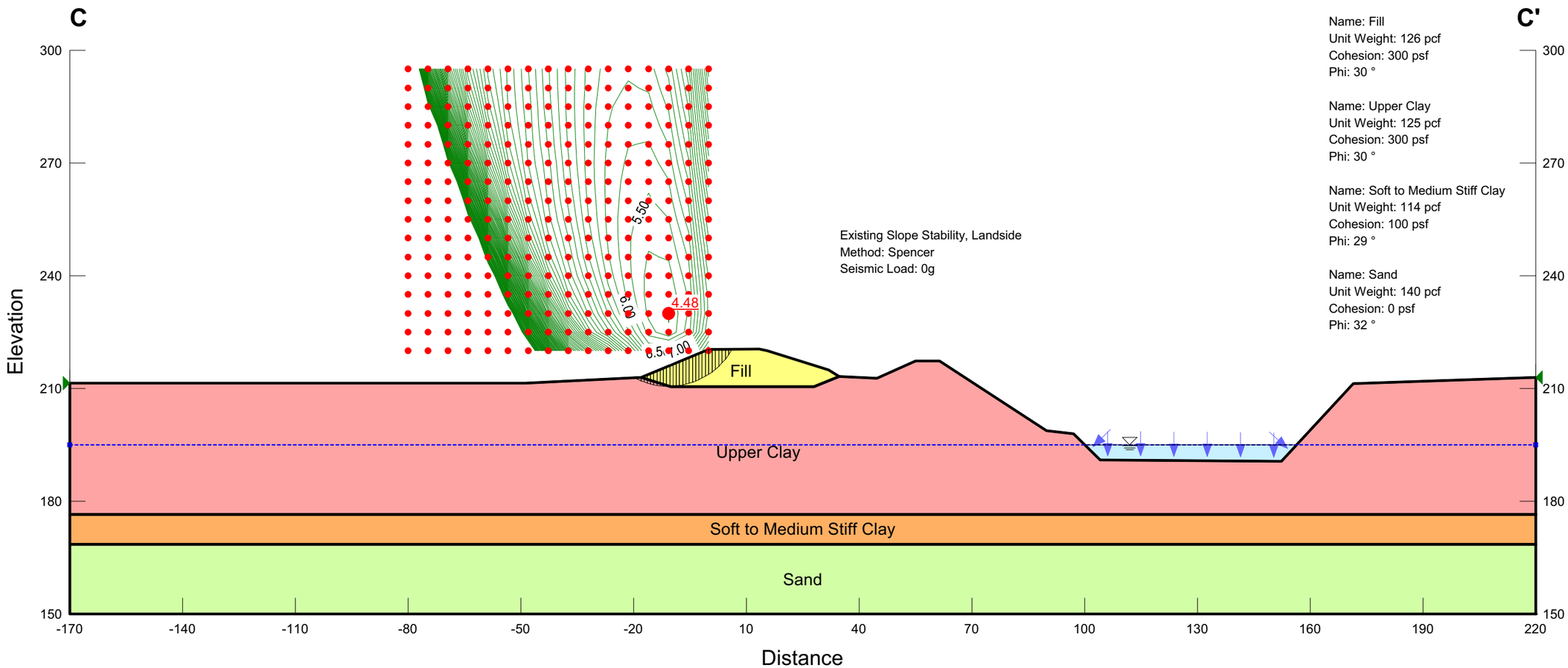


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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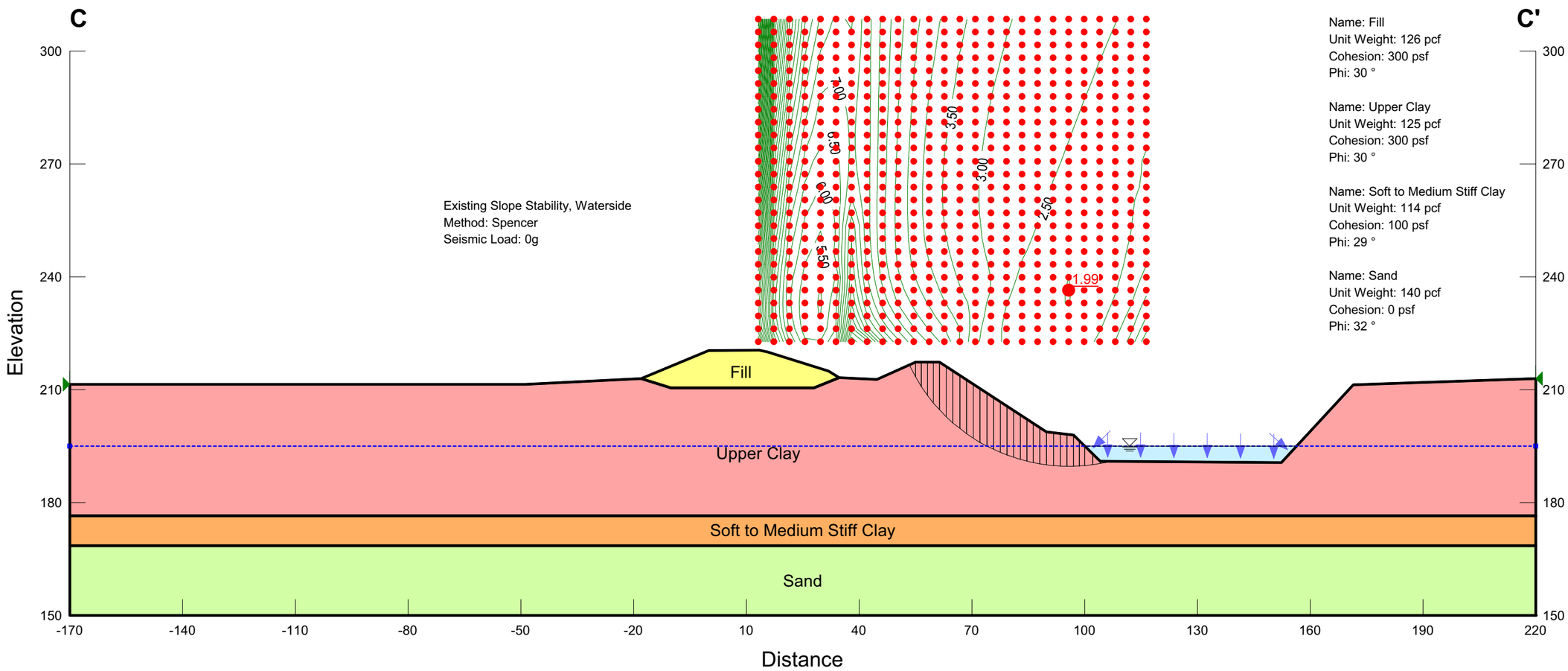


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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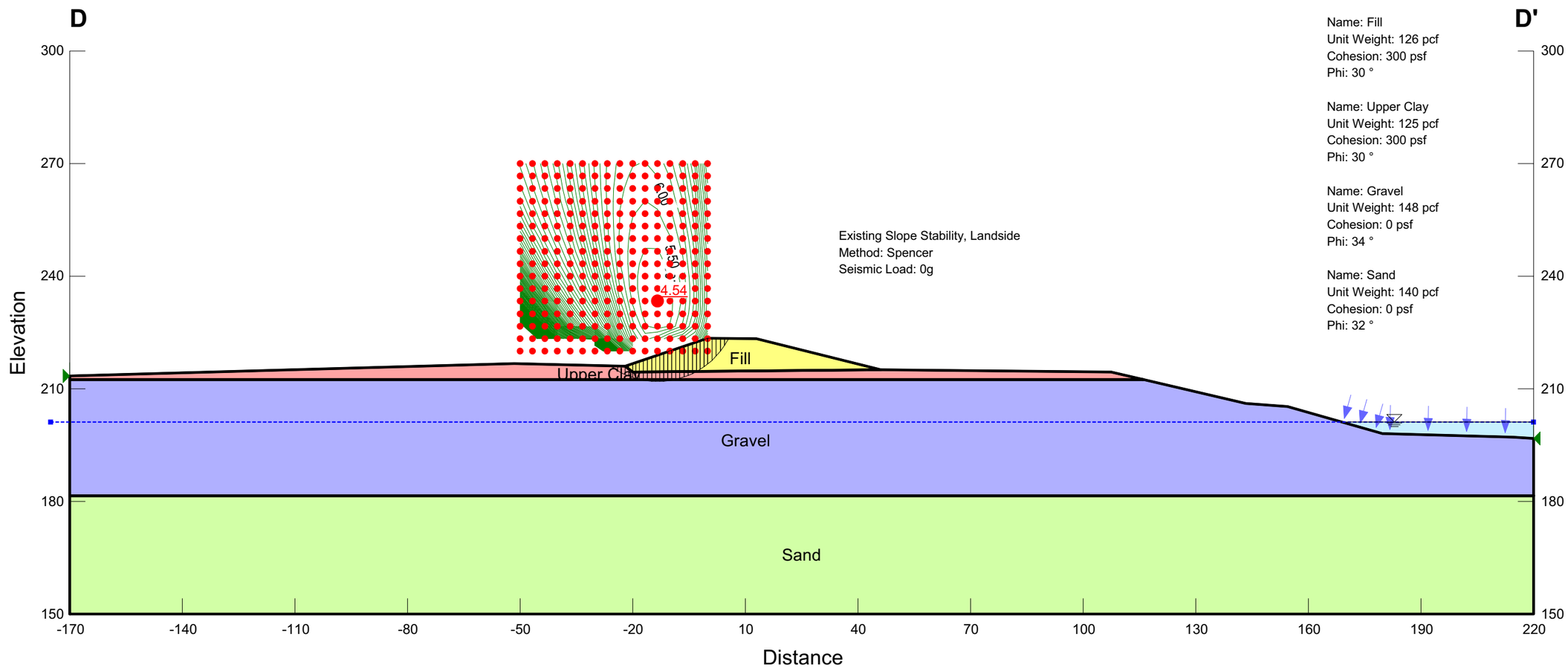


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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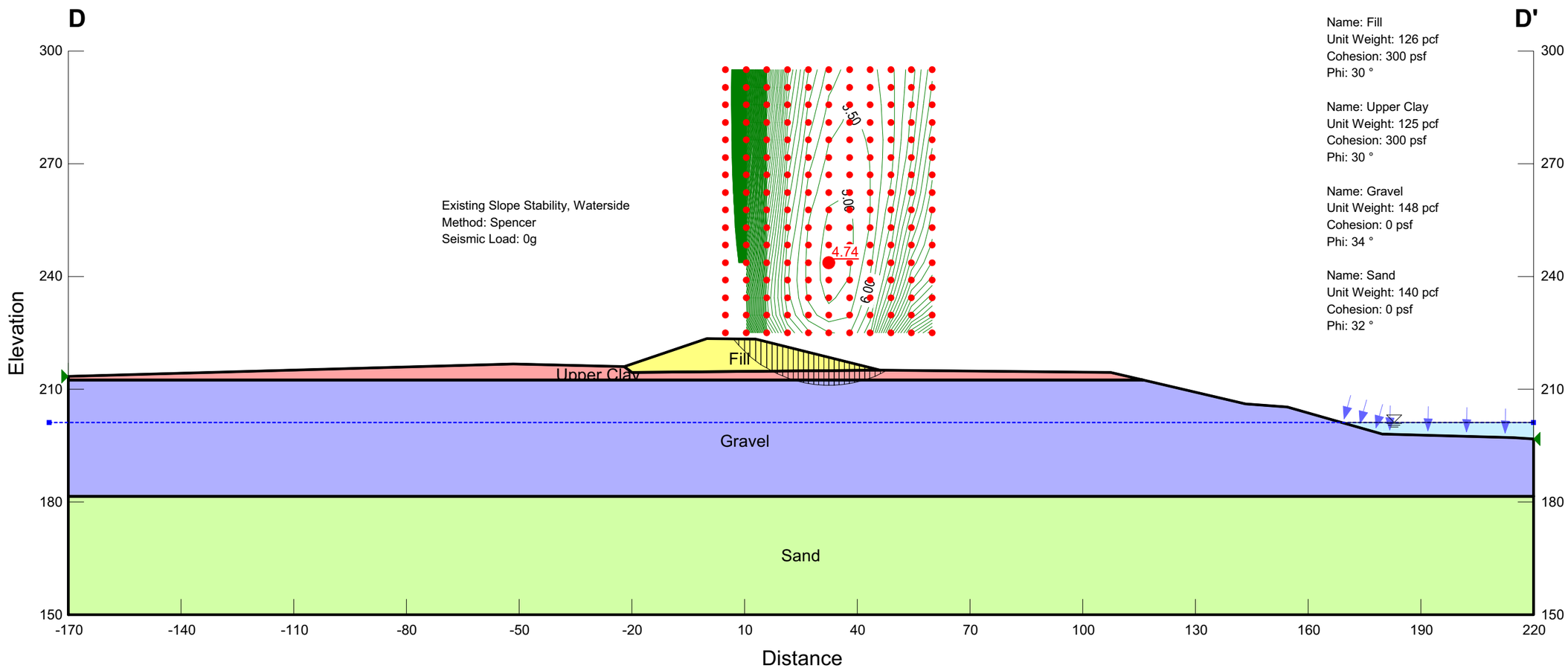


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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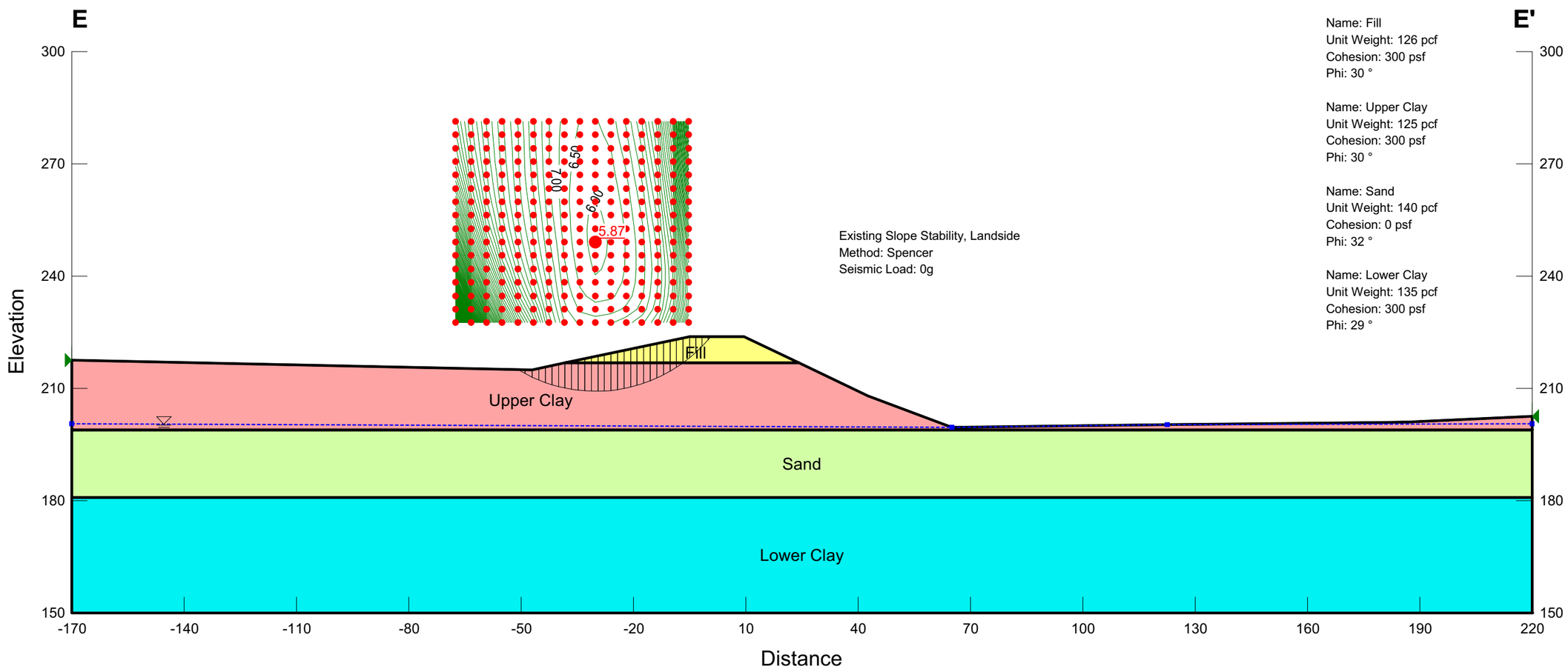


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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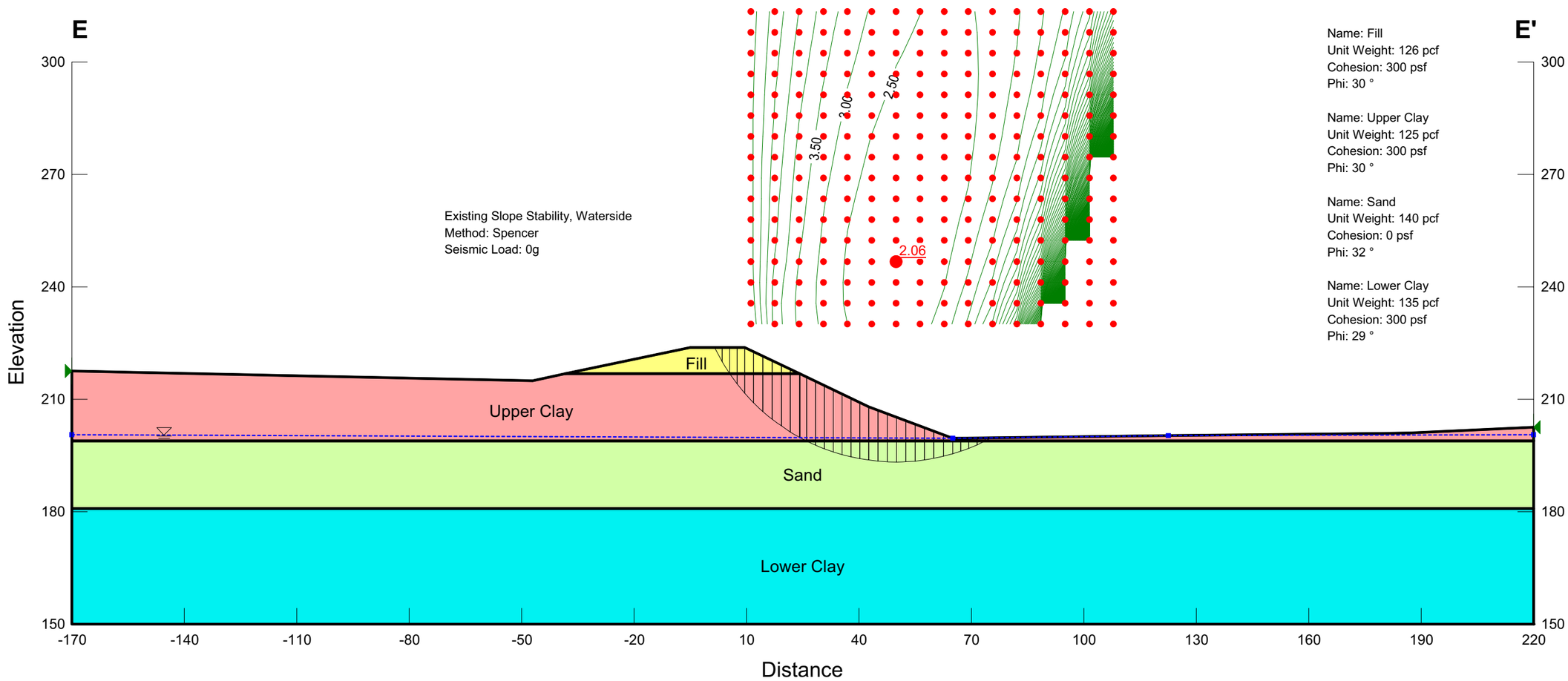


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Existing Slope Stability Result**<br>Waterside<br>Cross Section E<br>Levee Station 90+91 | **Figure**<br>10-12 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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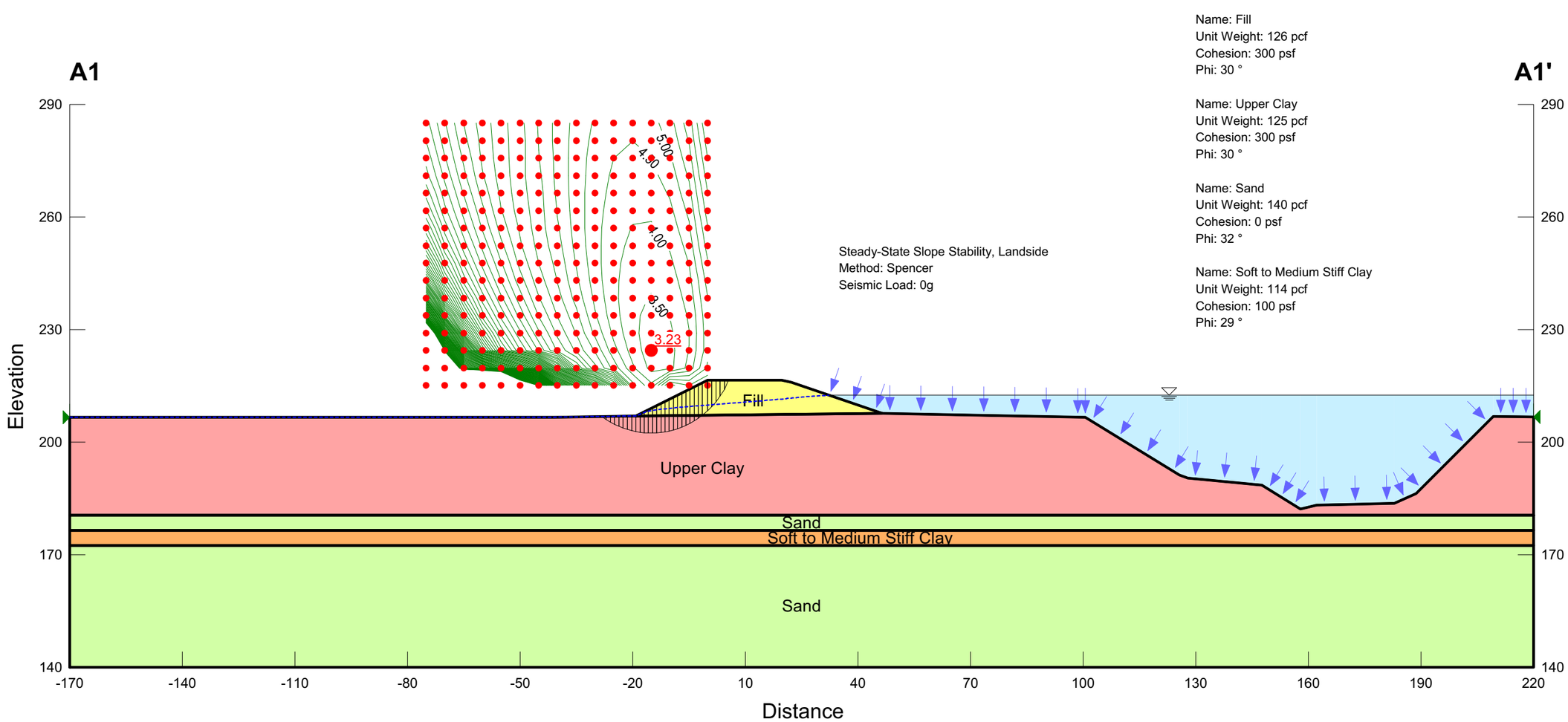


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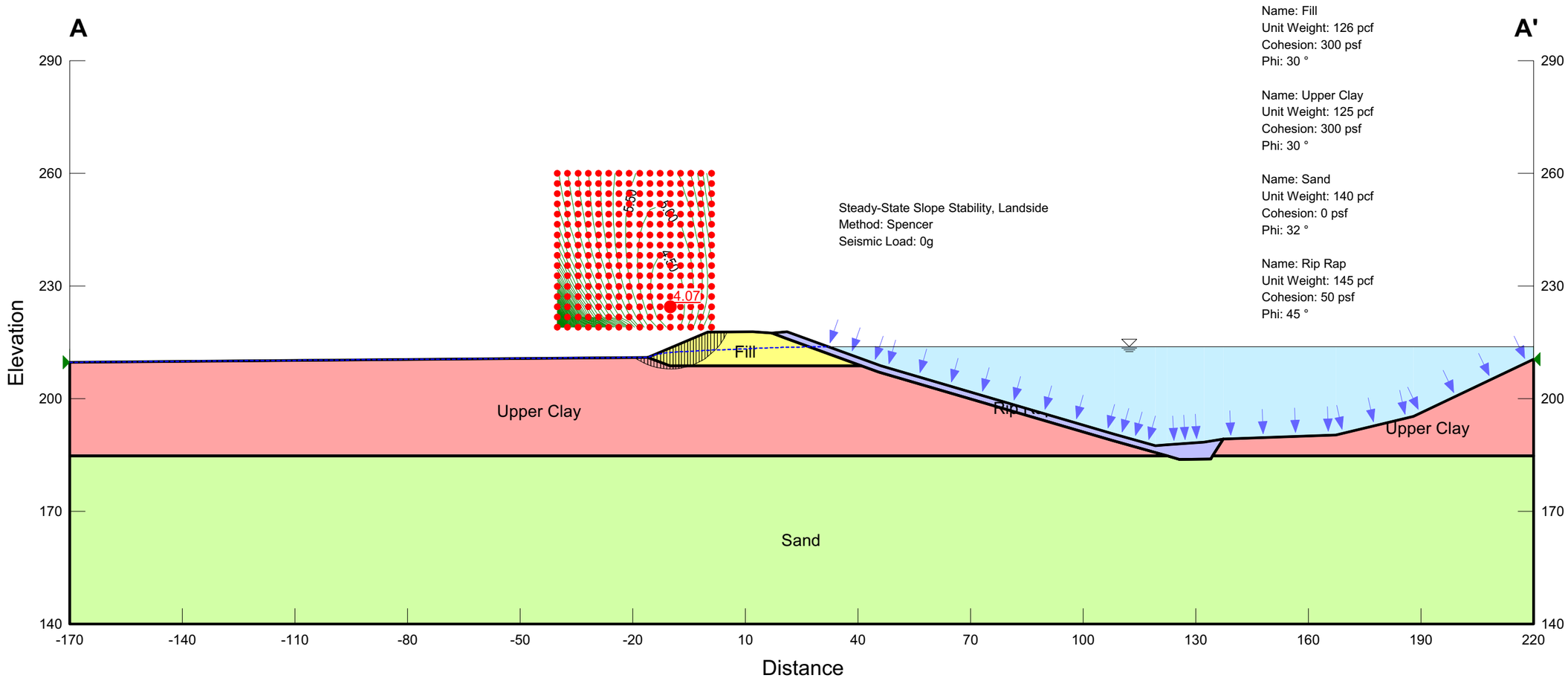


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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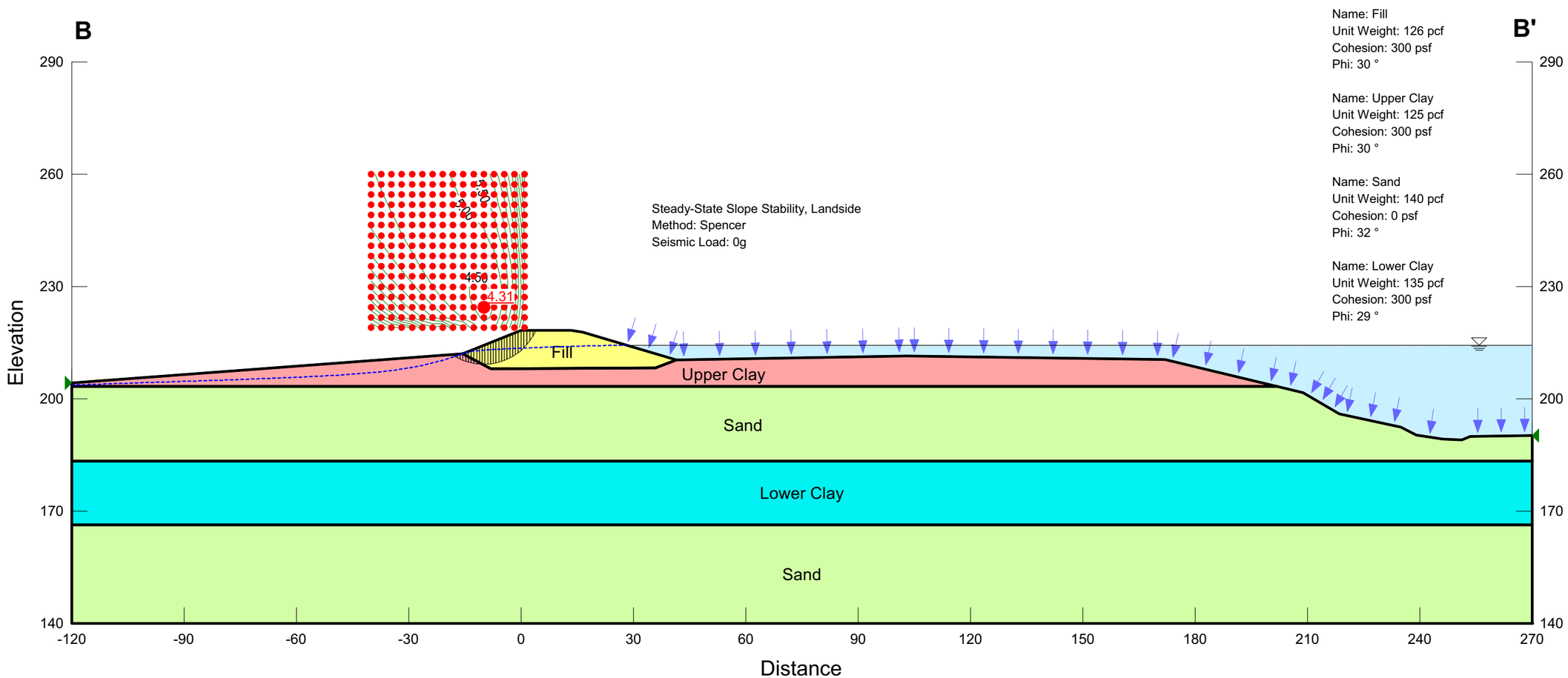


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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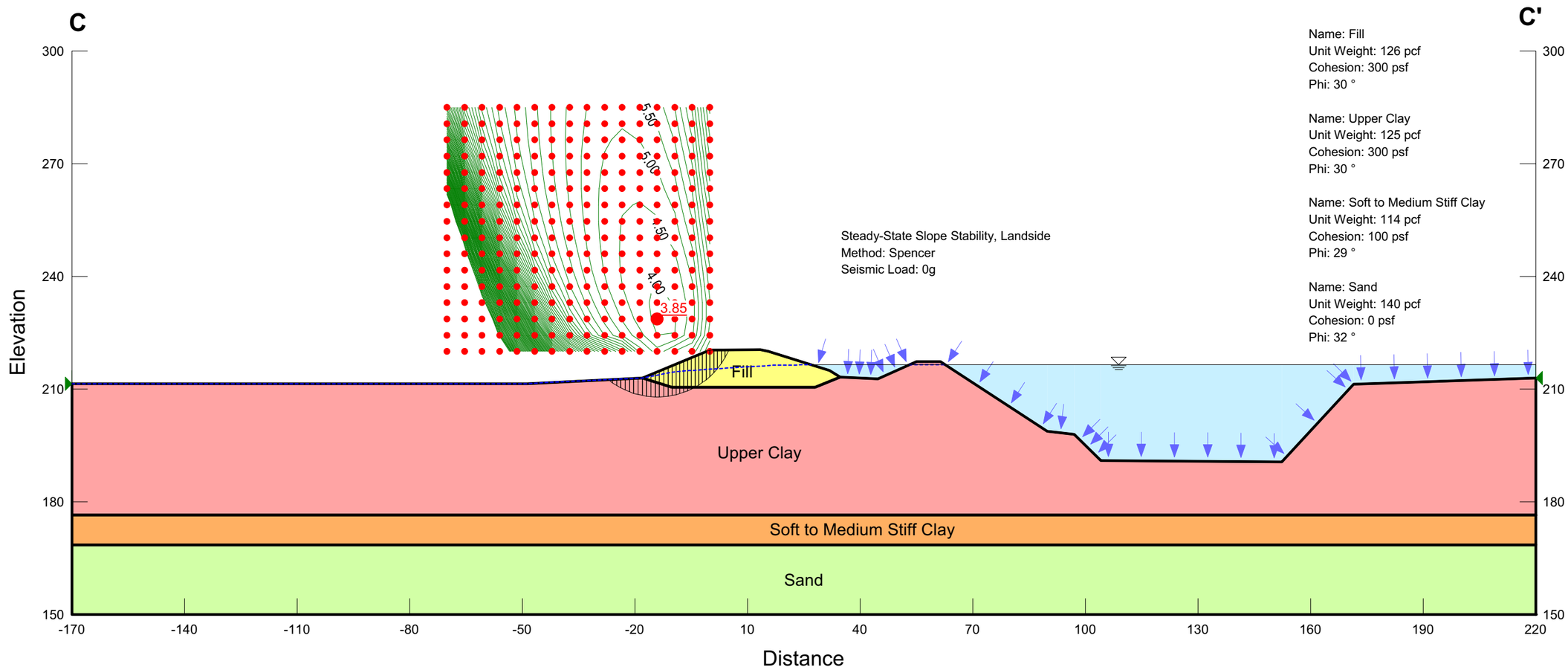


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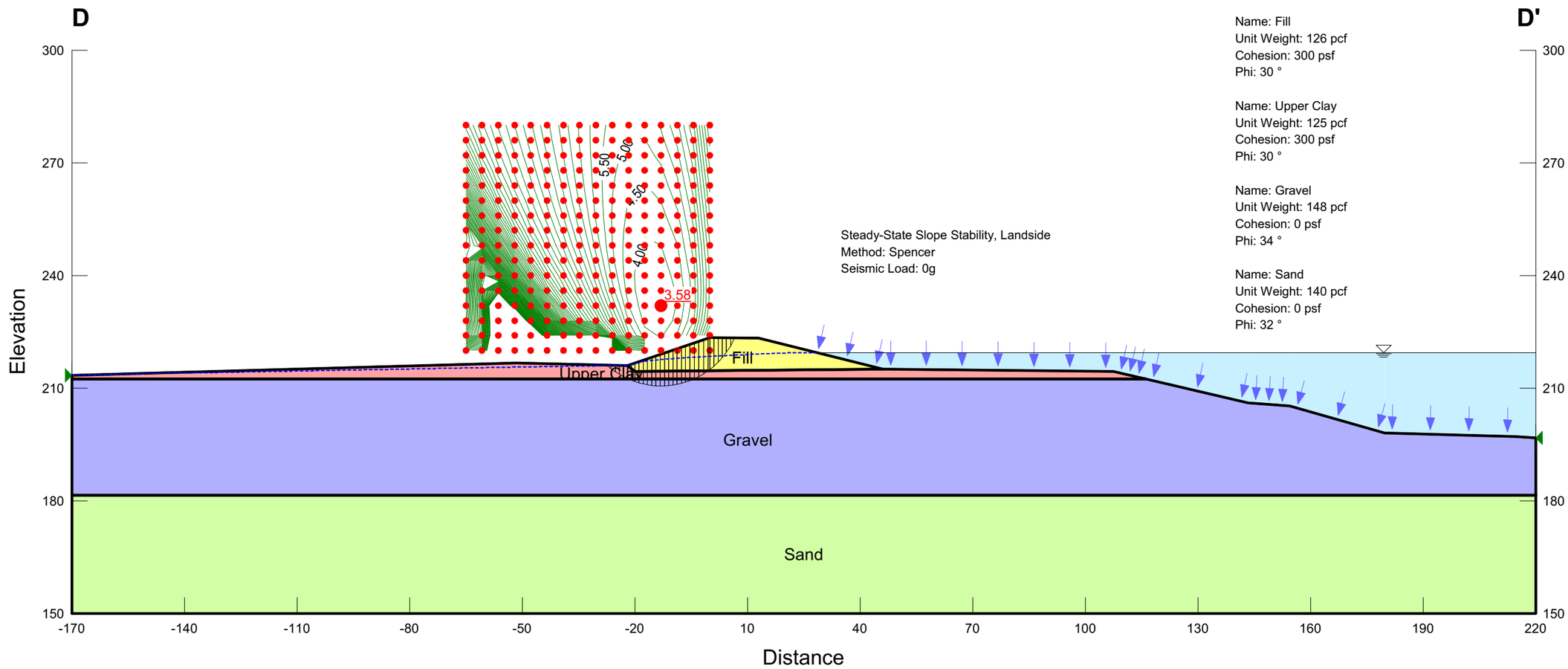


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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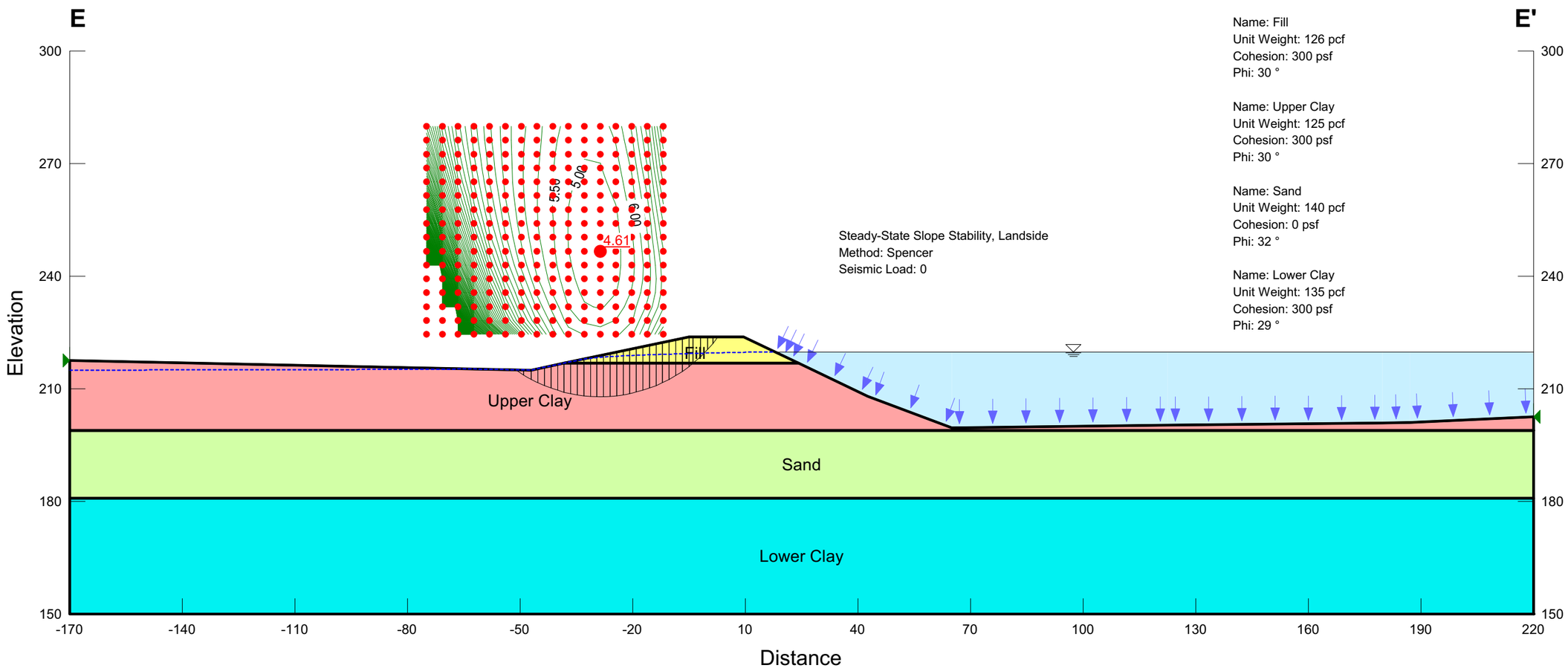


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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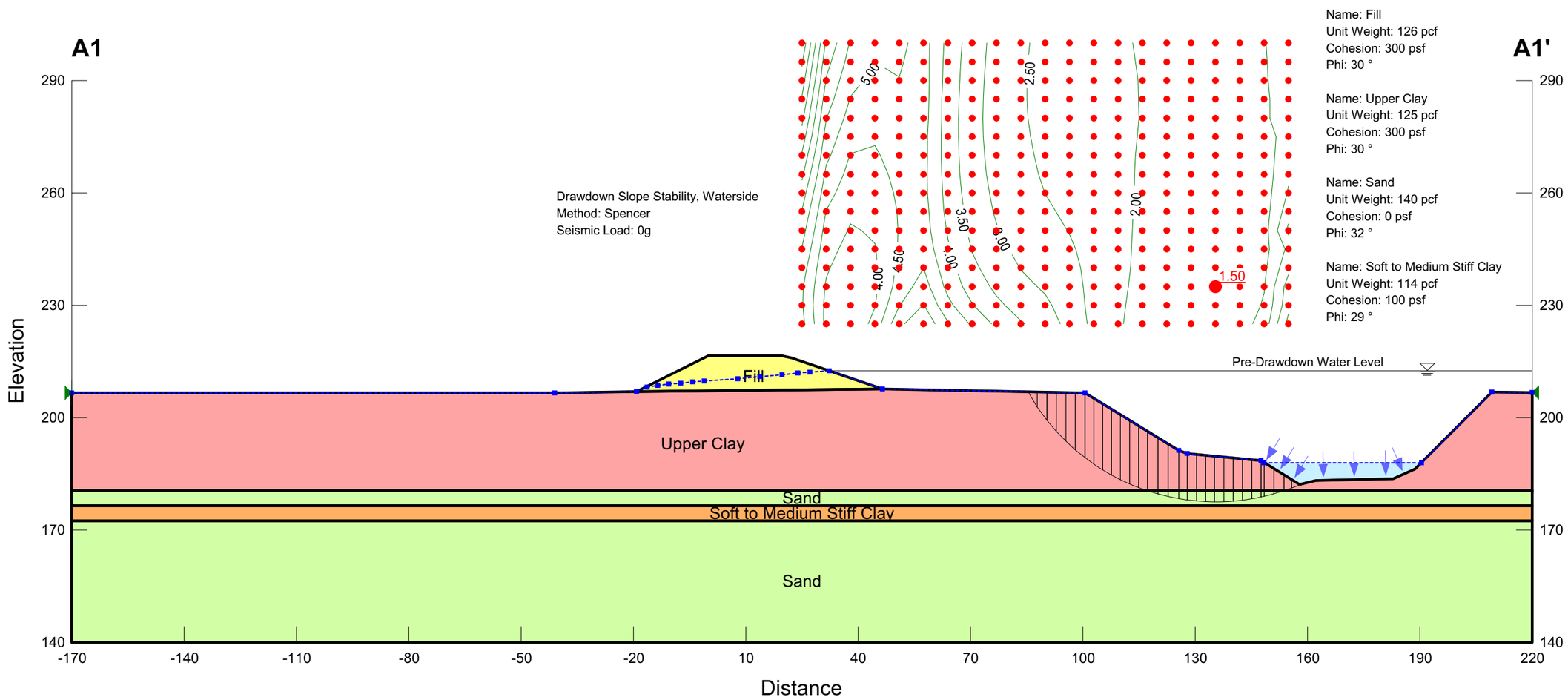


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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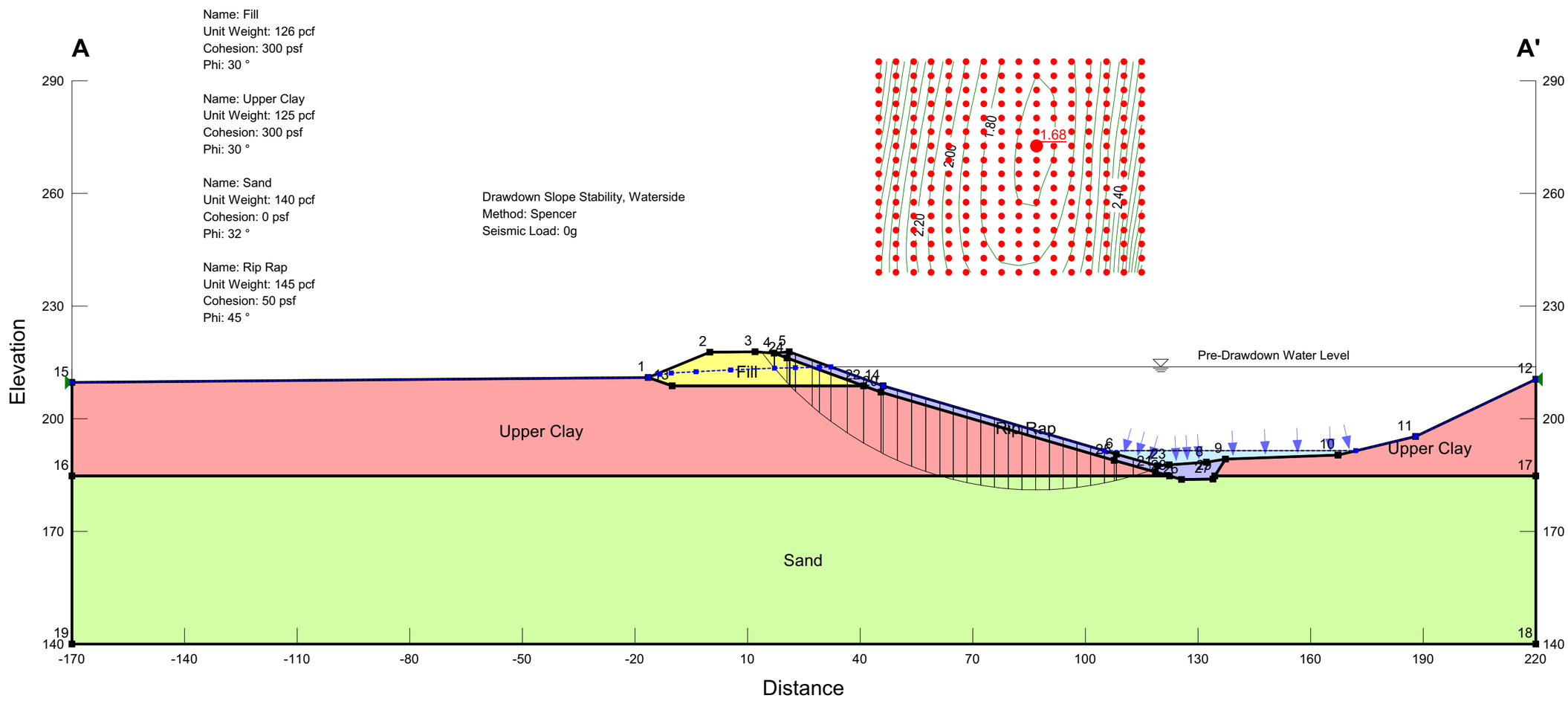


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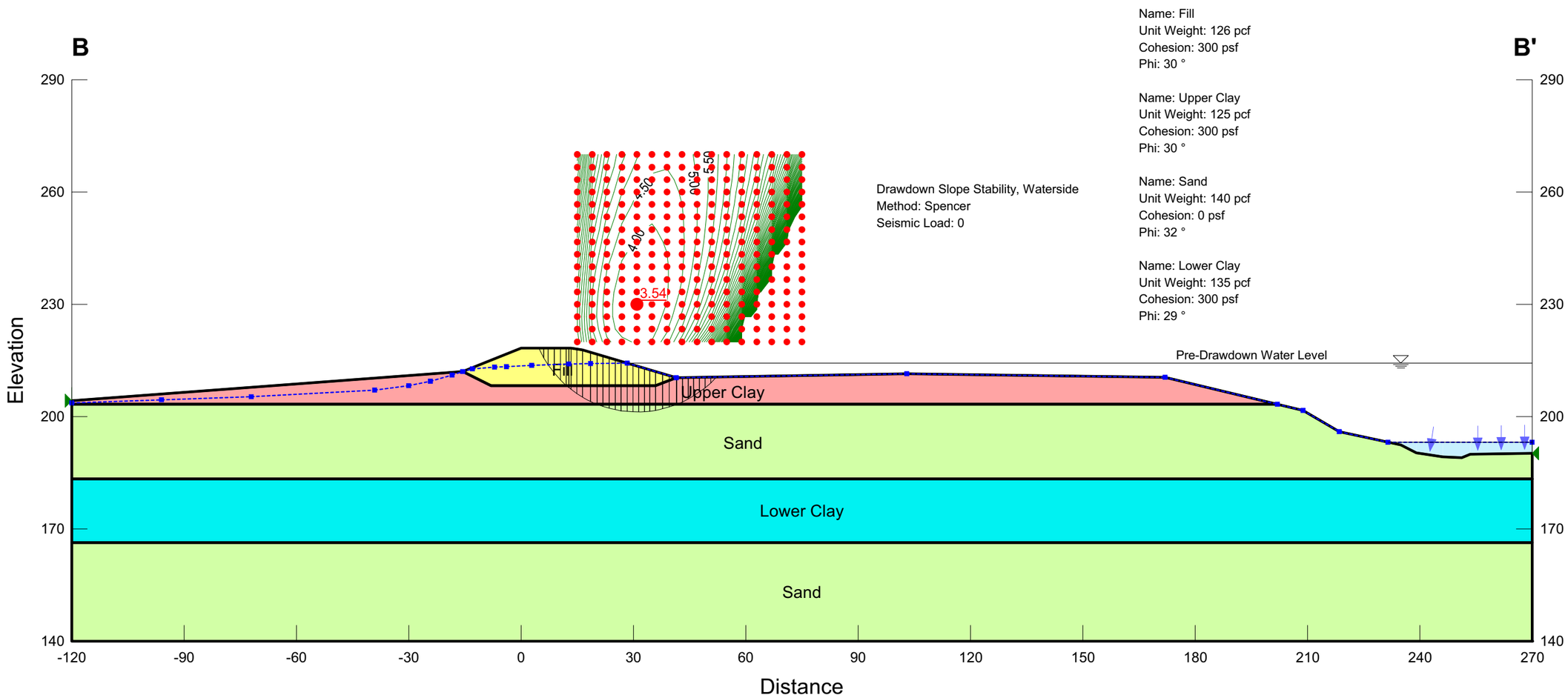
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Drawdown Slope Stability Result**<br>Waterside<br>Cross Section A<br>Levee Station 23+46 | **Figure**<br>12-2 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/4/2009 | Checked | | | |

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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Drawdown Slope Stability Result**<br>Waterside<br>Cross Section B<br>Levee Station 31+69 | **Figure**<br>12-3 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/28/2009 | Checked | | | |

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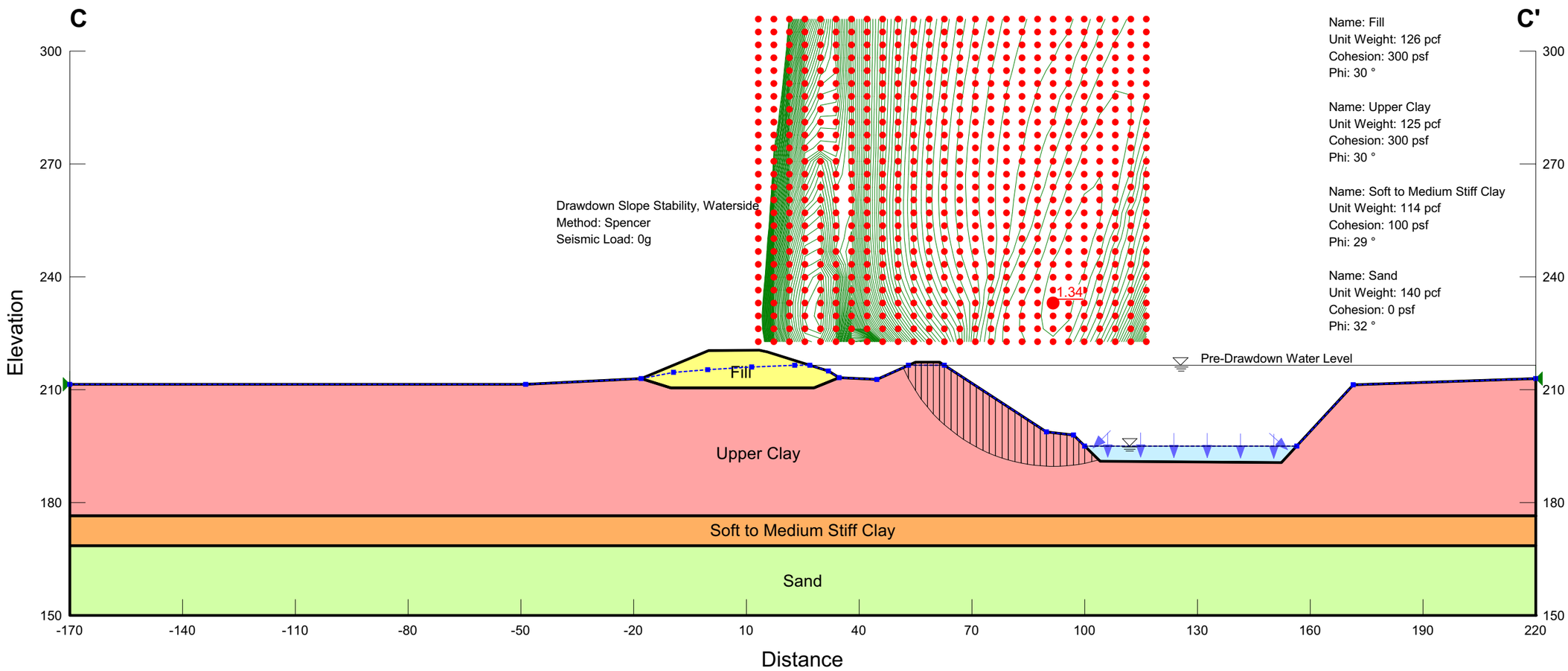


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Drawdown Slope Stability Result**<br>Waterside<br>Cross Section C<br>Levee Station 48+16 | **Figure**<br>12-4 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

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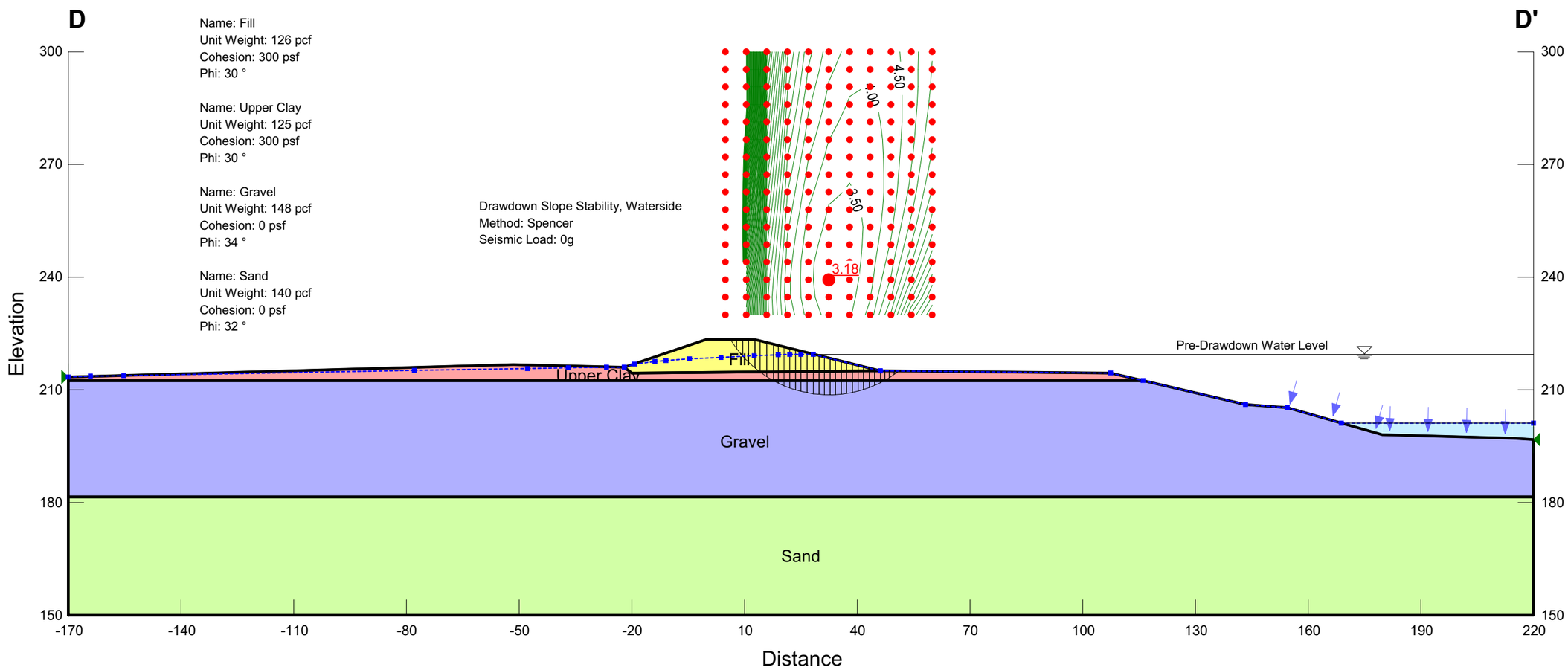


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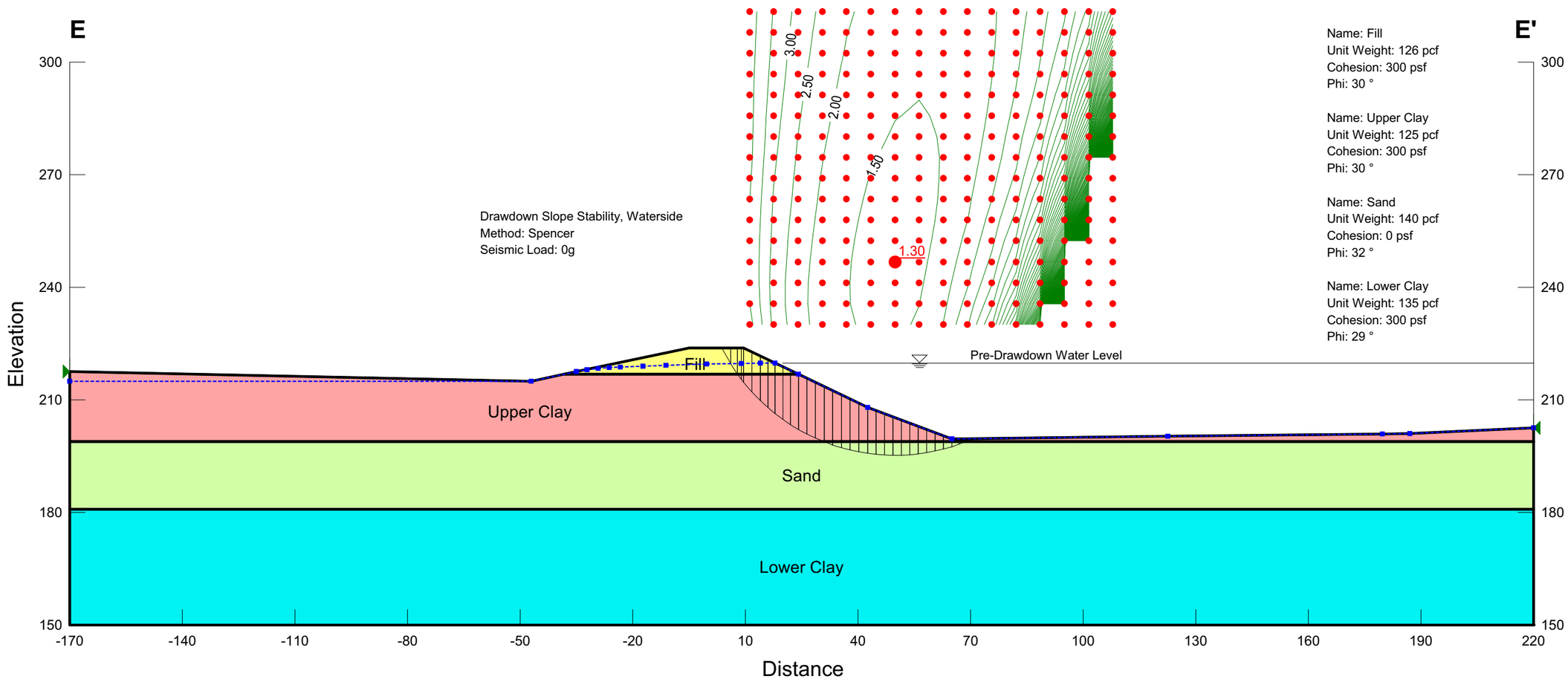
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Drawdown Slope Stability Result**<br>Waterside<br>Cross Section D<br>Levee Station 72+62 | **Figure**<br>12-5 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/29/2009 | Checked | | | |

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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/29/2009 | Checked | | | |

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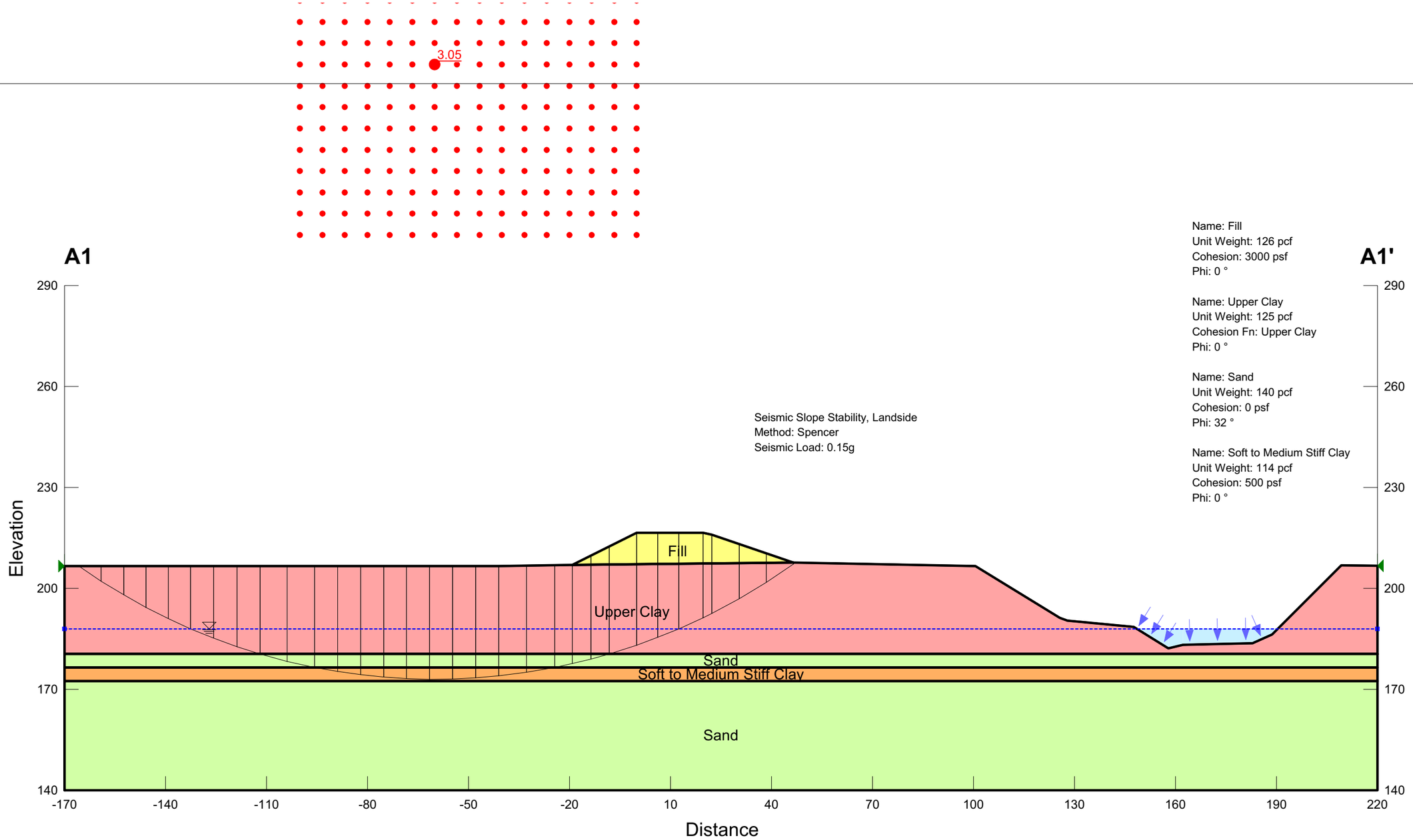


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

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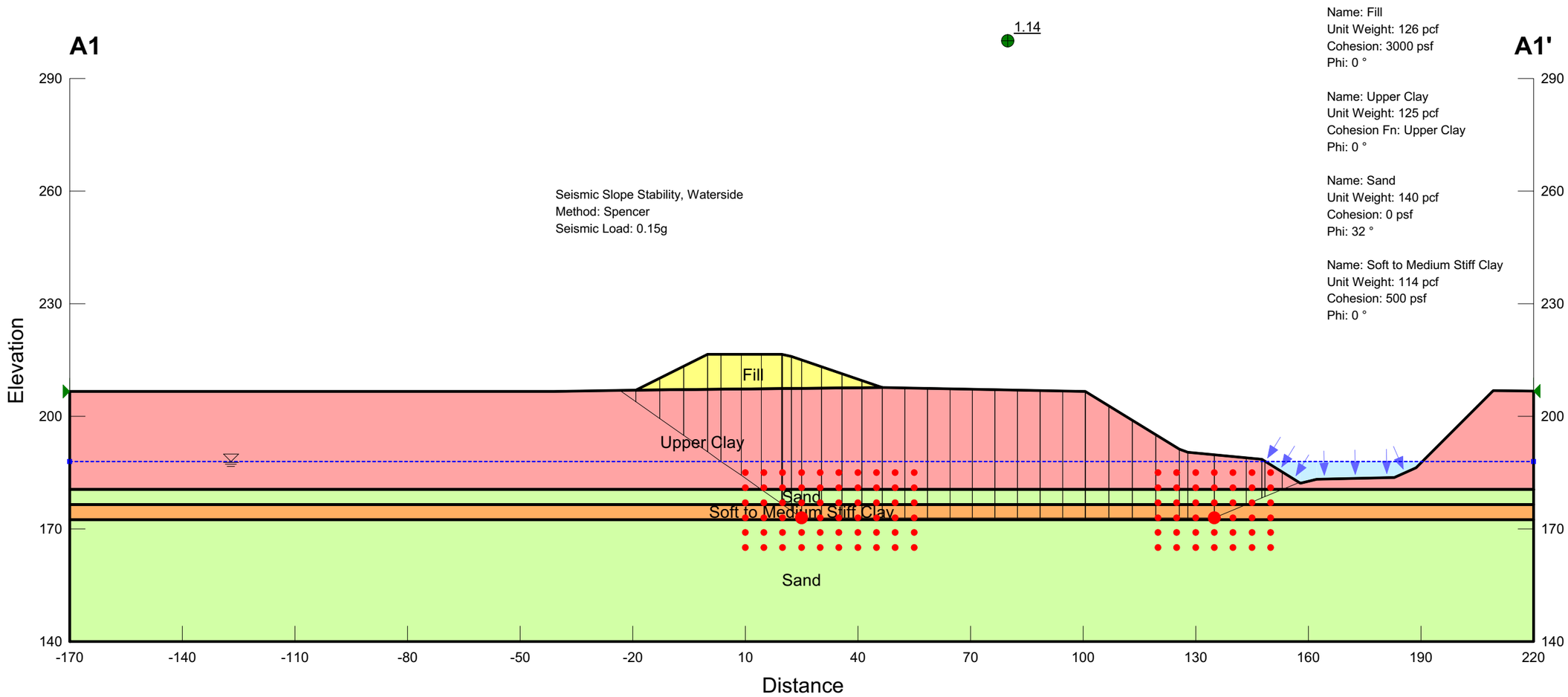


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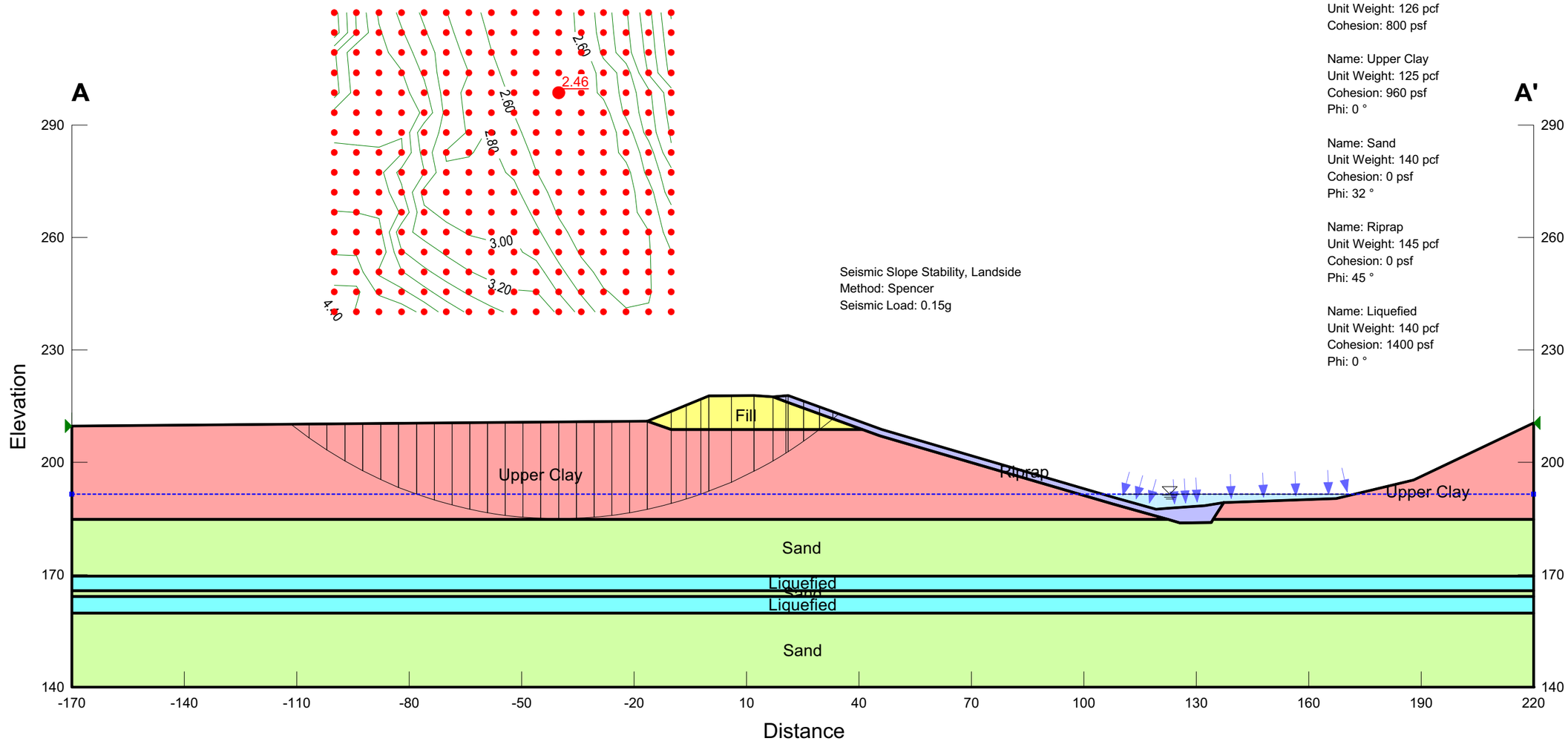
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Screening Analysis**<br>Waterside<br>Cross Section A1<br>Levee Station 11+06 | **Figure**<br>13-2 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Screening Analysis**<br>Landside<br>Cross Section A<br>Levee Station 23+46 | **Figure**<br>13-3 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/7/2009 | Checked | | | |

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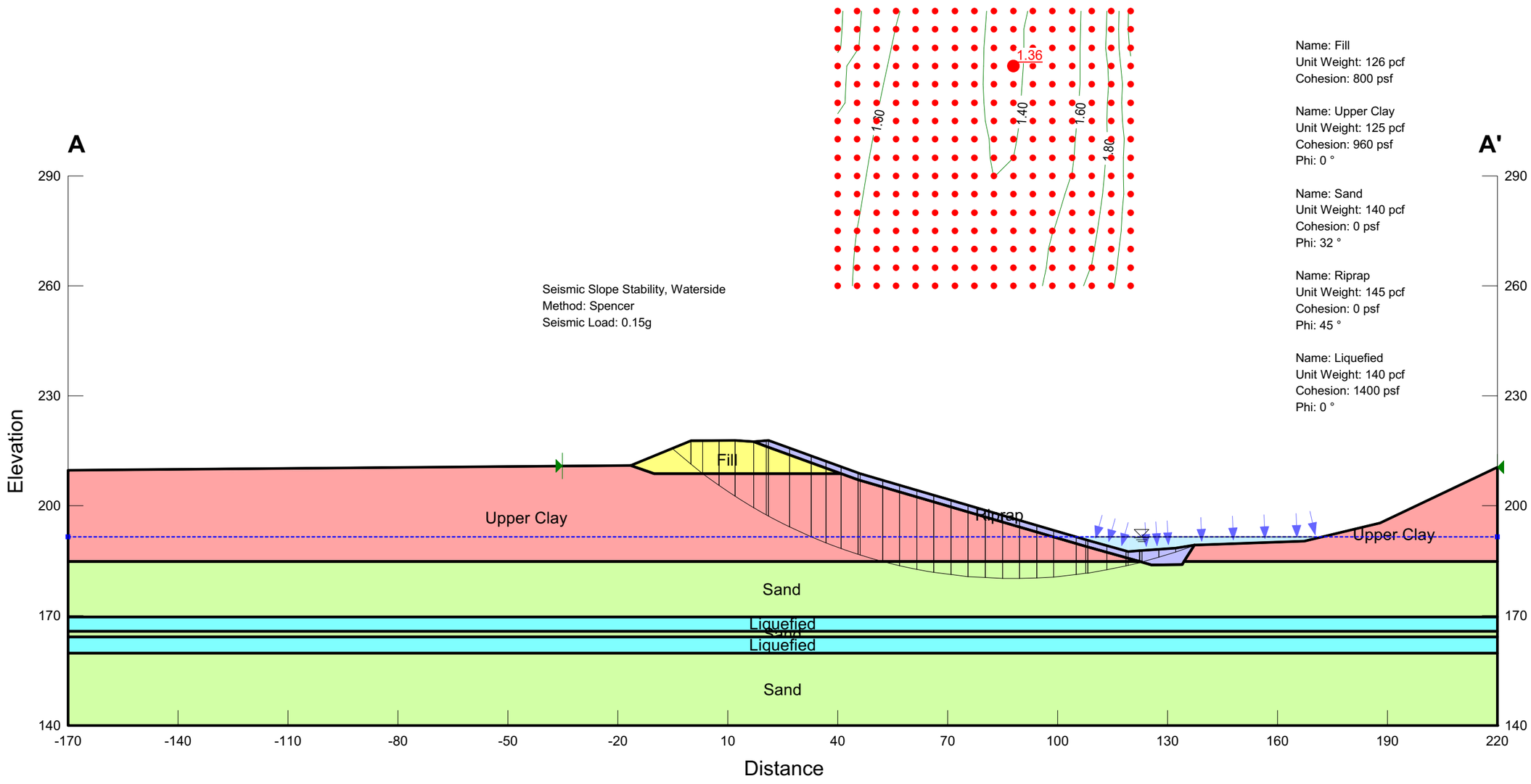


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Screening Analysis**<br>Waterside<br>Cross Section A<br>Levee Station 23+46 | **Figure**<br>13-4 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/7/2009 | Checked | | | |

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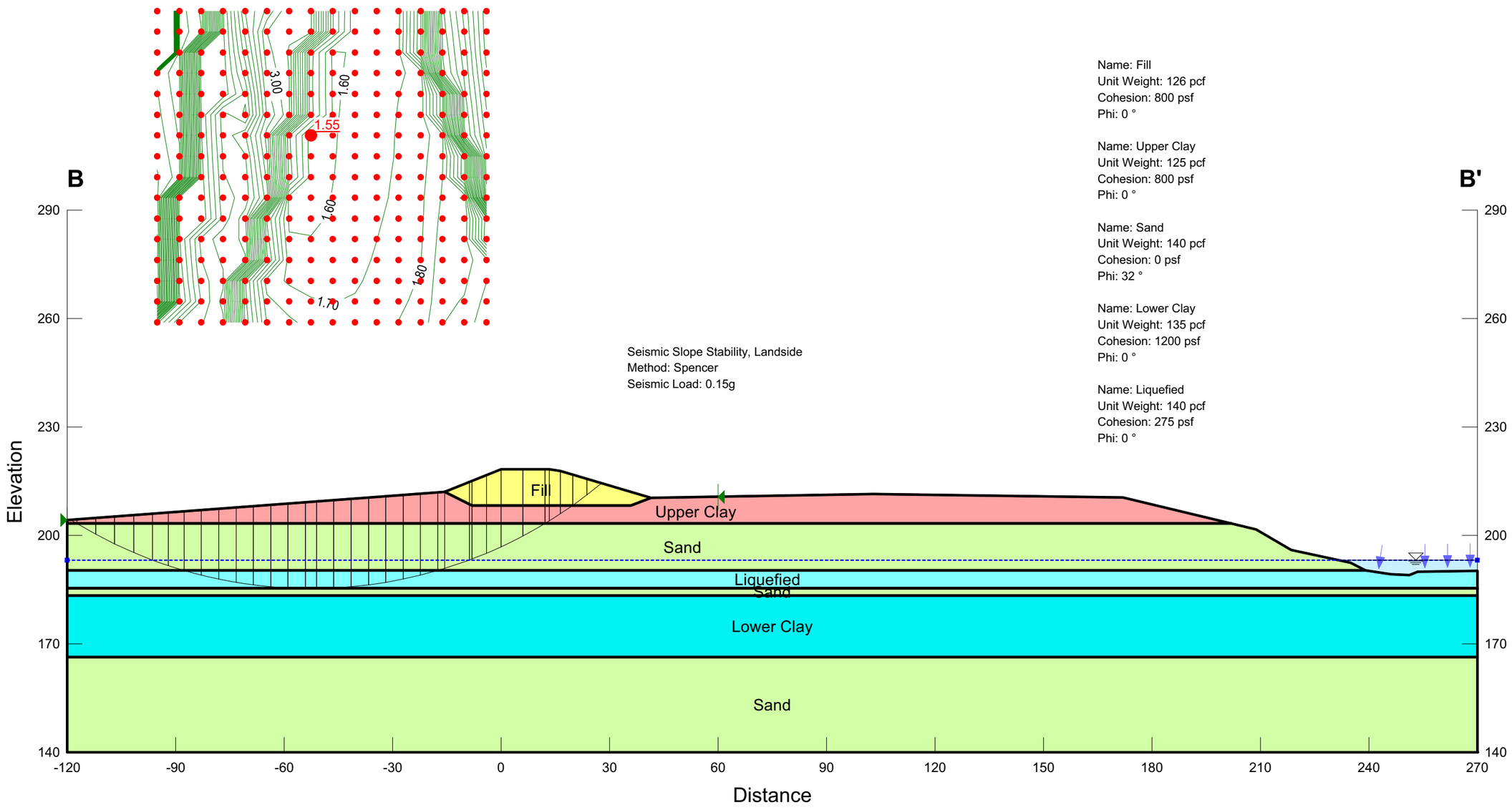


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Screening Analysis**<br>Landside<br>Cross Section B<br>Levee Station 31+69 | **Figure**<br>13-5 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/7/2009 | Checked | | | |

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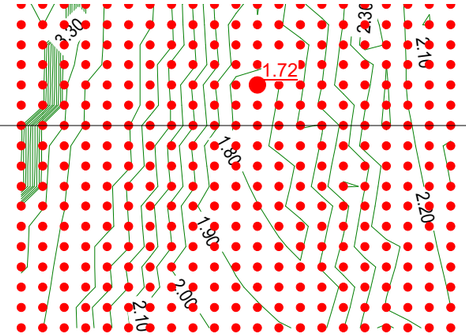


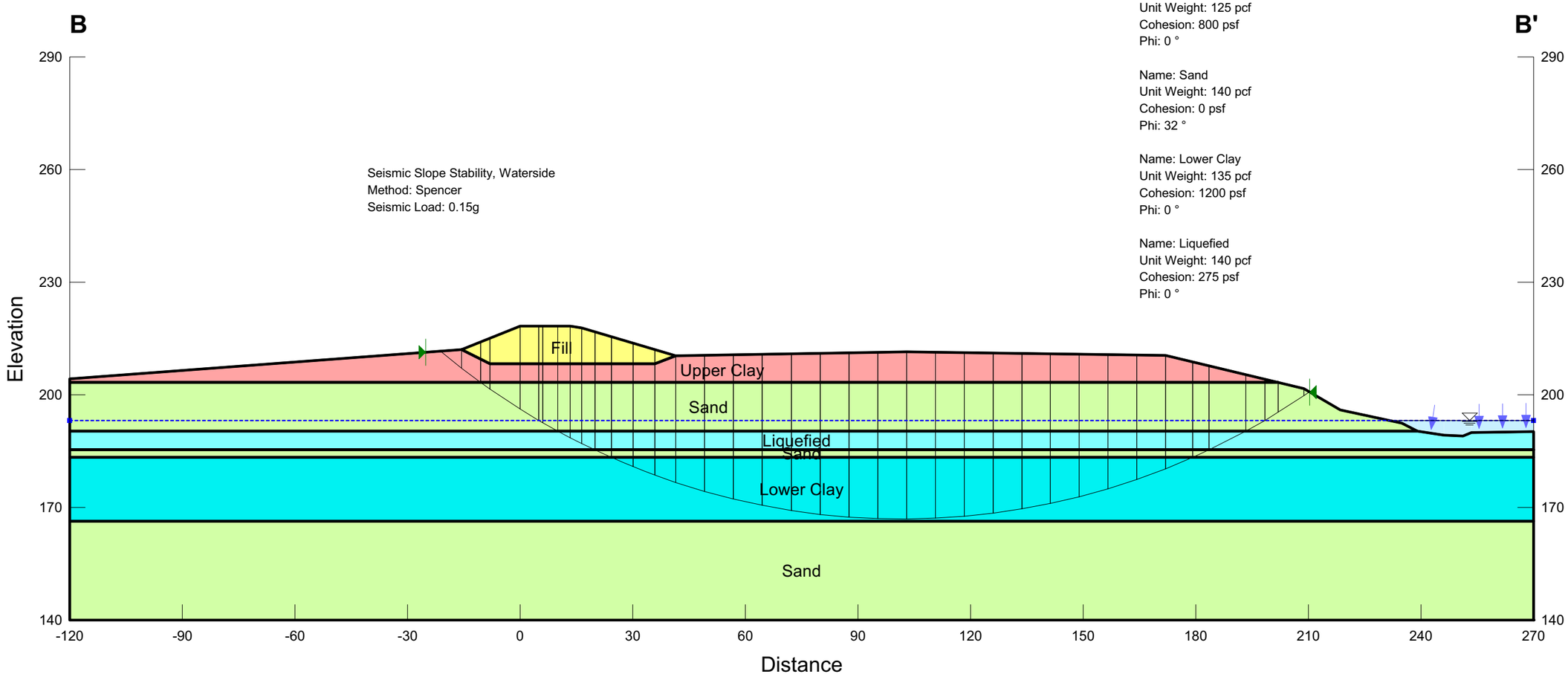

Name: Fill
Unit Weight: 126 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Upper Clay
Unit Weight: 125 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Sand
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 32 °

Name: Lower Clay
Unit Weight: 135 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Liquefied
Unit Weight: 140 pcf
Cohesion: 275 psf
Phi: 0 °

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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Screening Analysis**<br>Waterside<br>Cross Section B<br>Levee Station 31+69 | **Figure**<br>13-6 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/7/2009 | Checked | | | |

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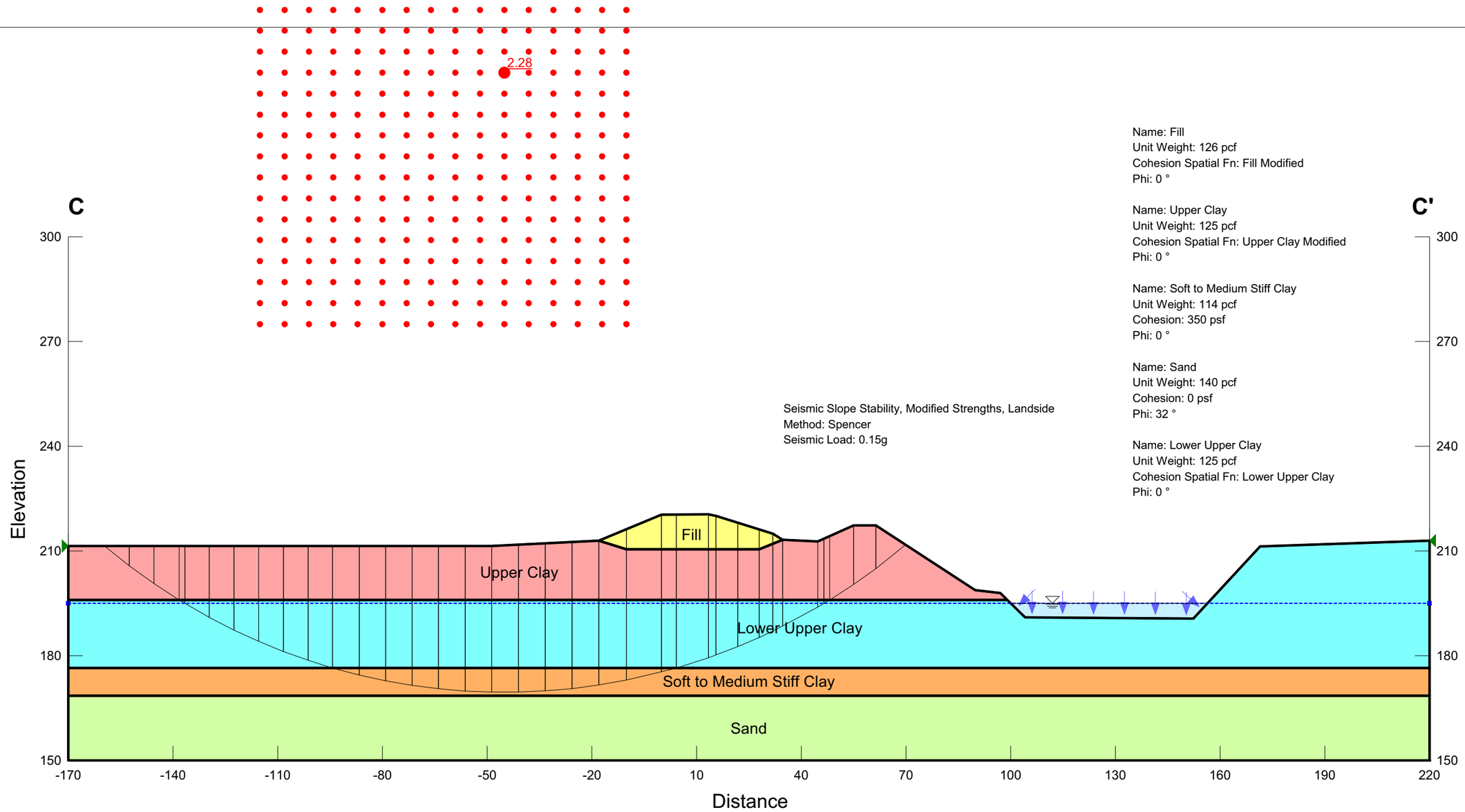


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Screening Analysis**<br>Landside<br>Cross Section C<br>Levee Station 48+16 | **Figure**<br>13-7 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

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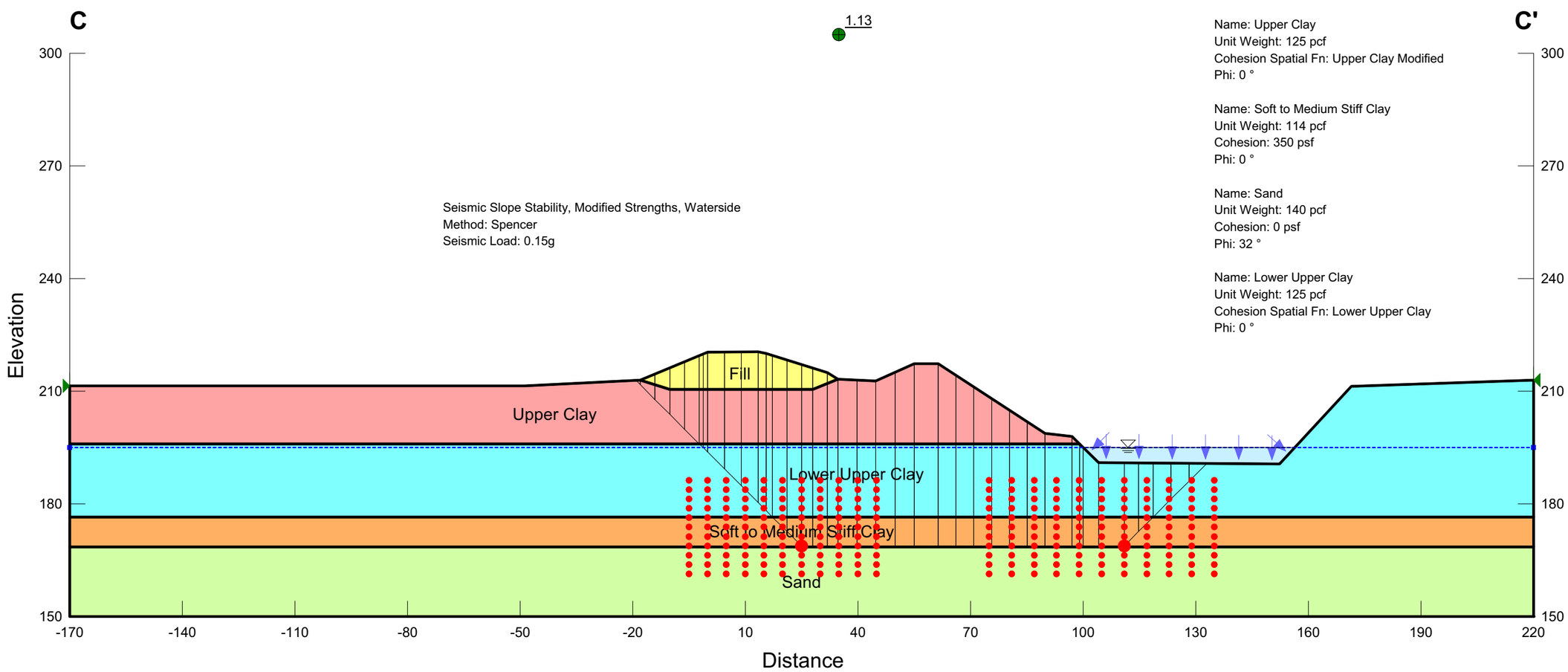


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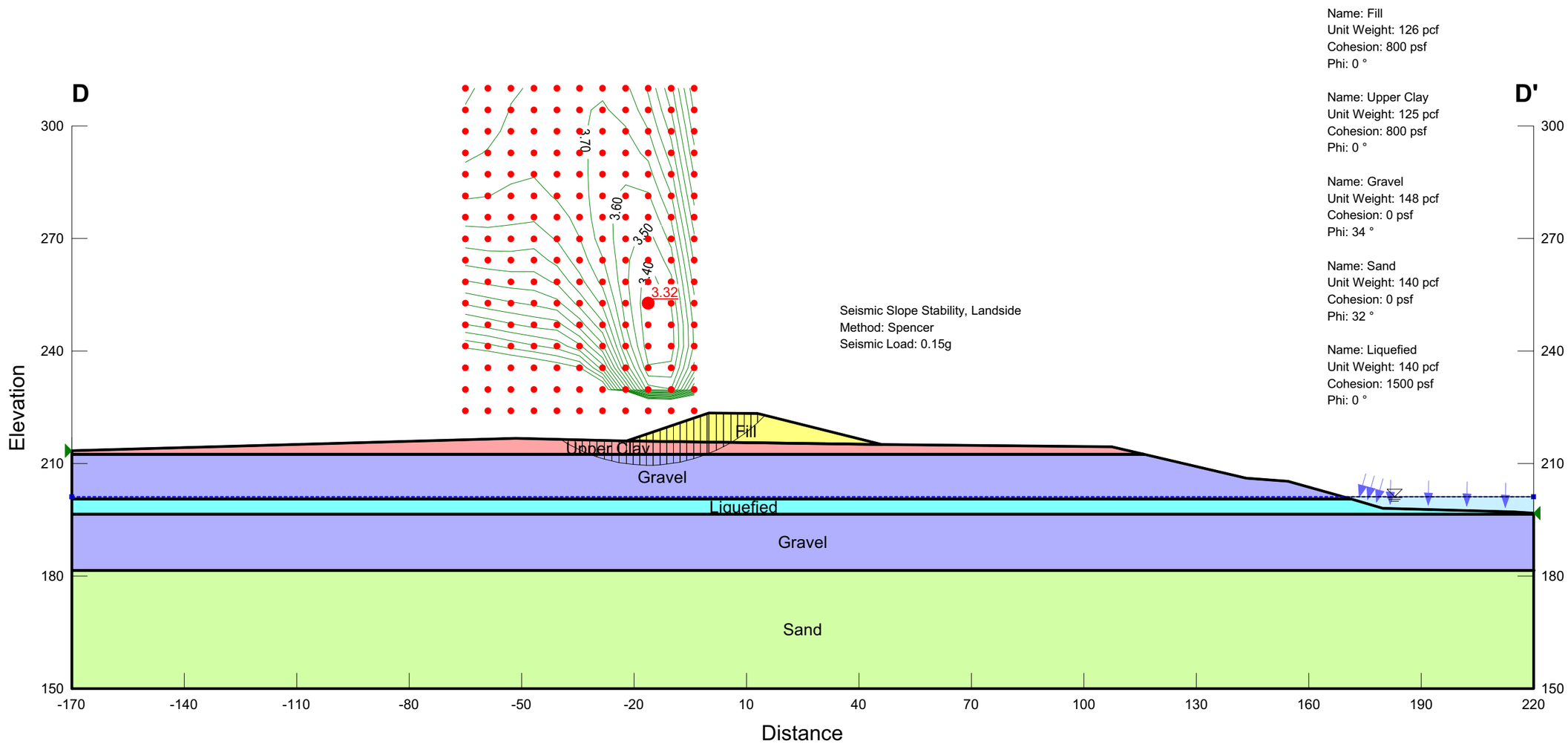
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Screening Analysis**<br>Waterside<br>Cross Section C<br>Levee Station 48+16 | **Figure**<br>13-8 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/7/2009 | Checked | | | |

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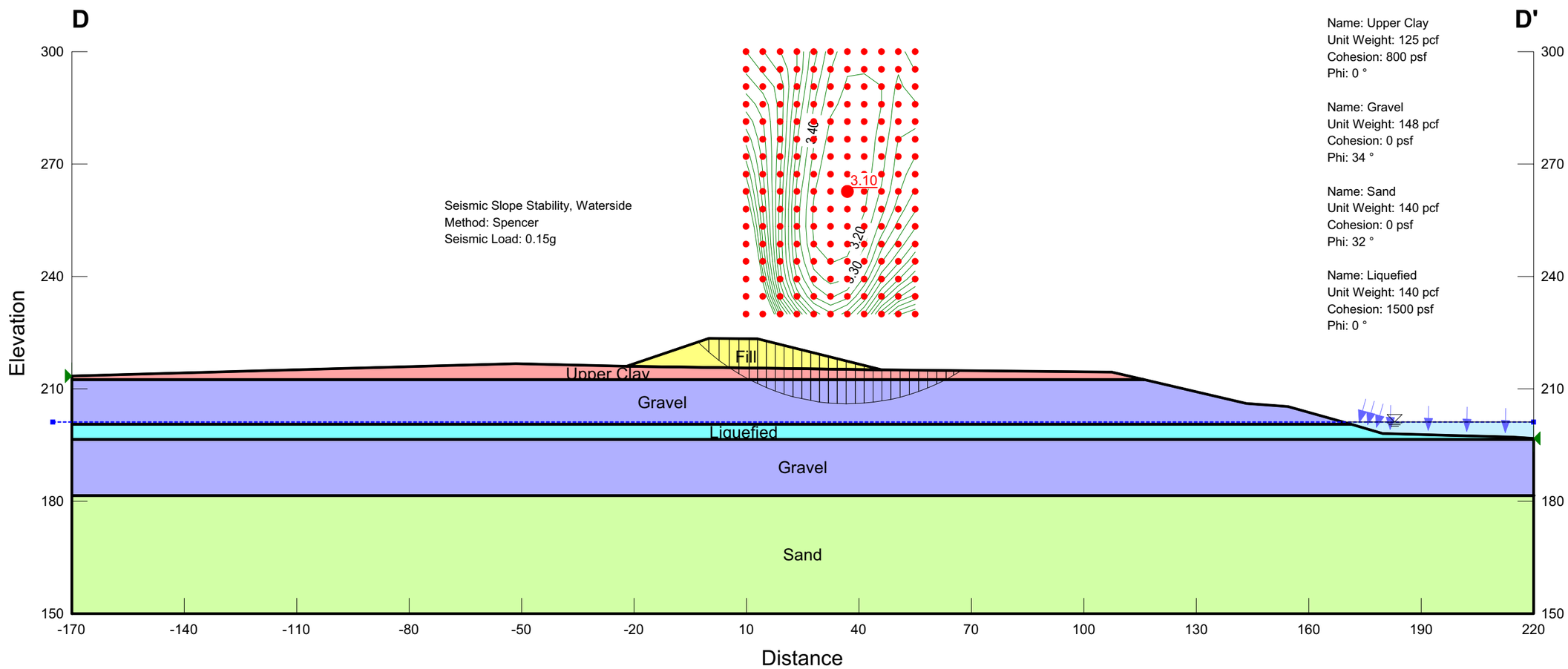


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Screening Analysis**<br>Waterside<br>Cross Section D<br>Levee Station 72+62 | **Figure**<br>13-10 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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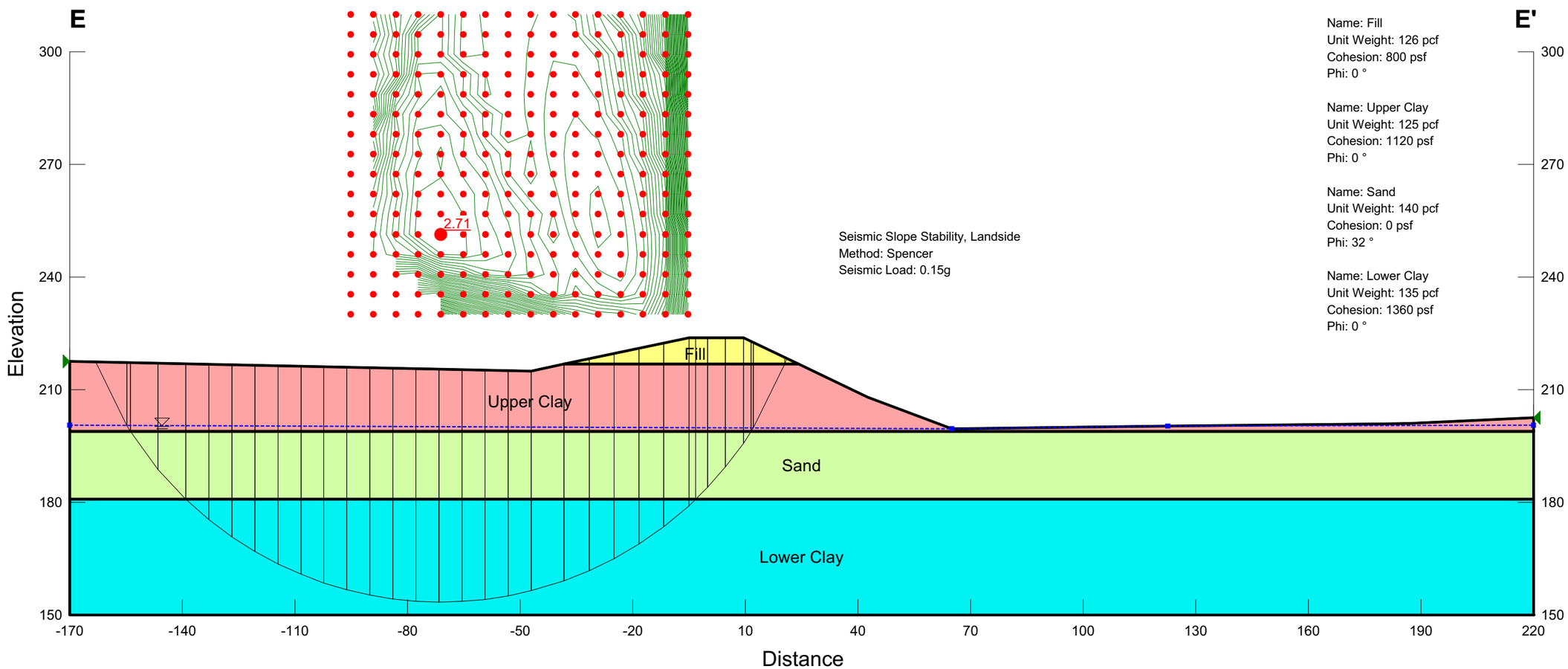


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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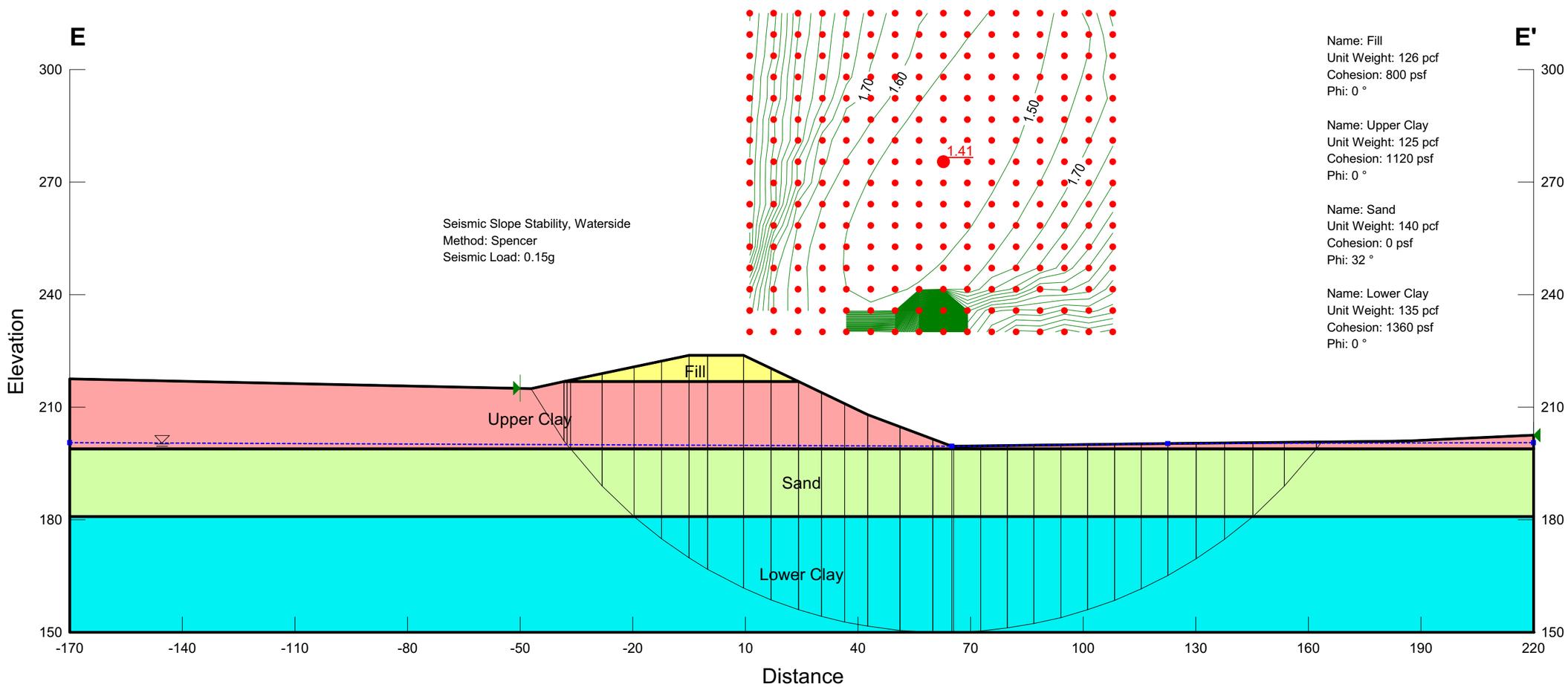


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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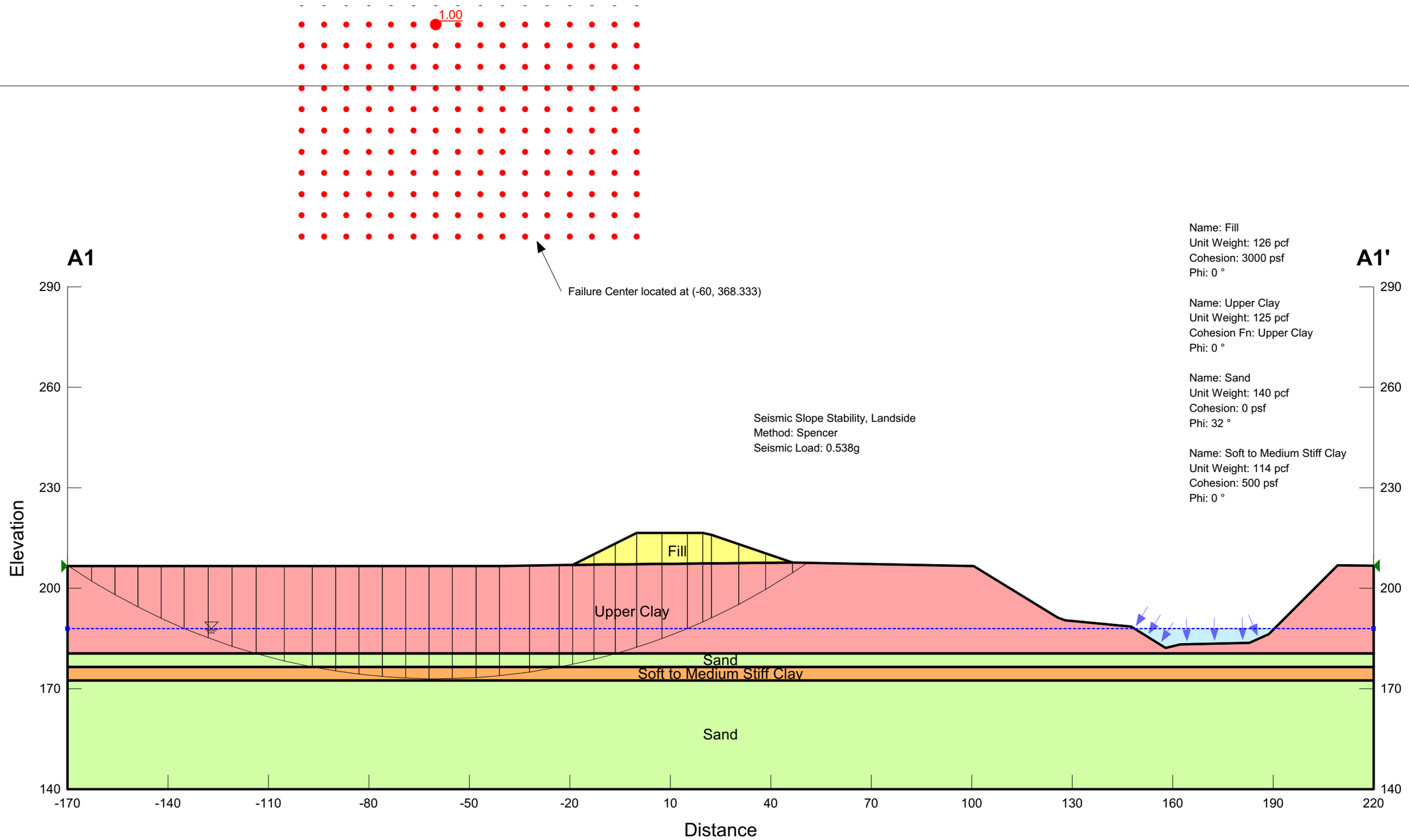


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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
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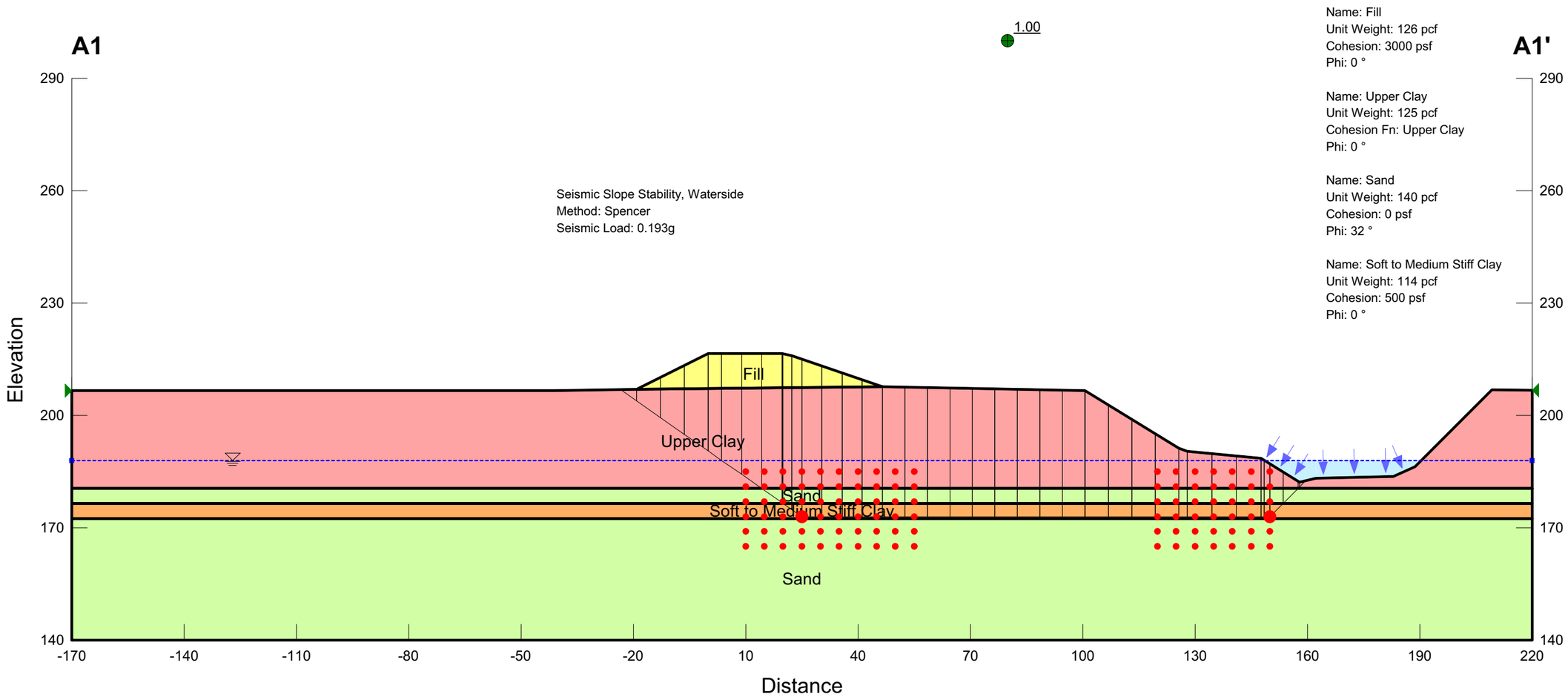


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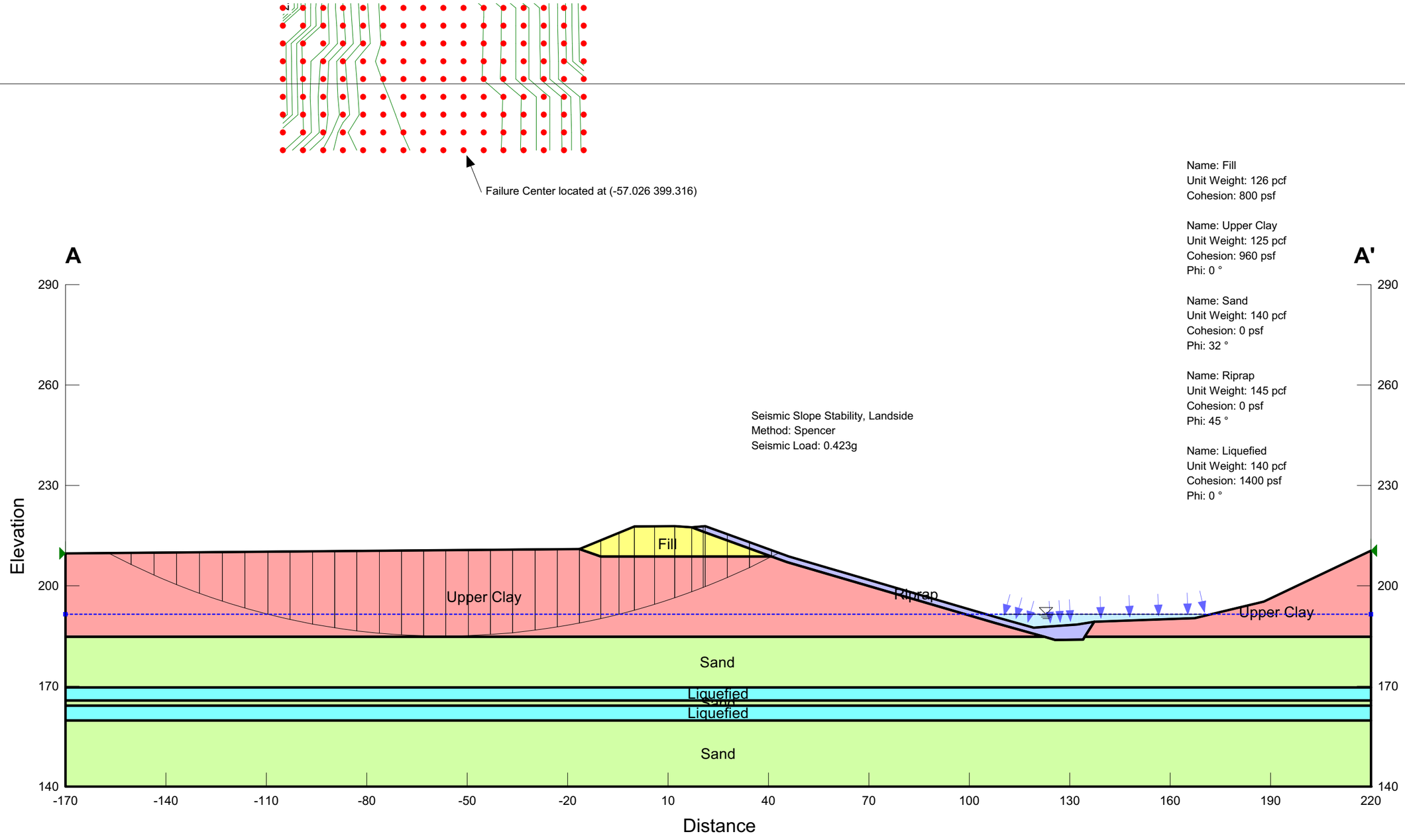
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

Directory: I:\Project\13000\13815.001\12000 CALC EVAL ANALYSES\Geo Studio\Section A1\A1_RS_20853_SeismicKy_block.gsz Date: 5/19/2009 Time: 11:08:48 AM Last Edited By: Levulel

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| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/7/2009 | Checked | | | |

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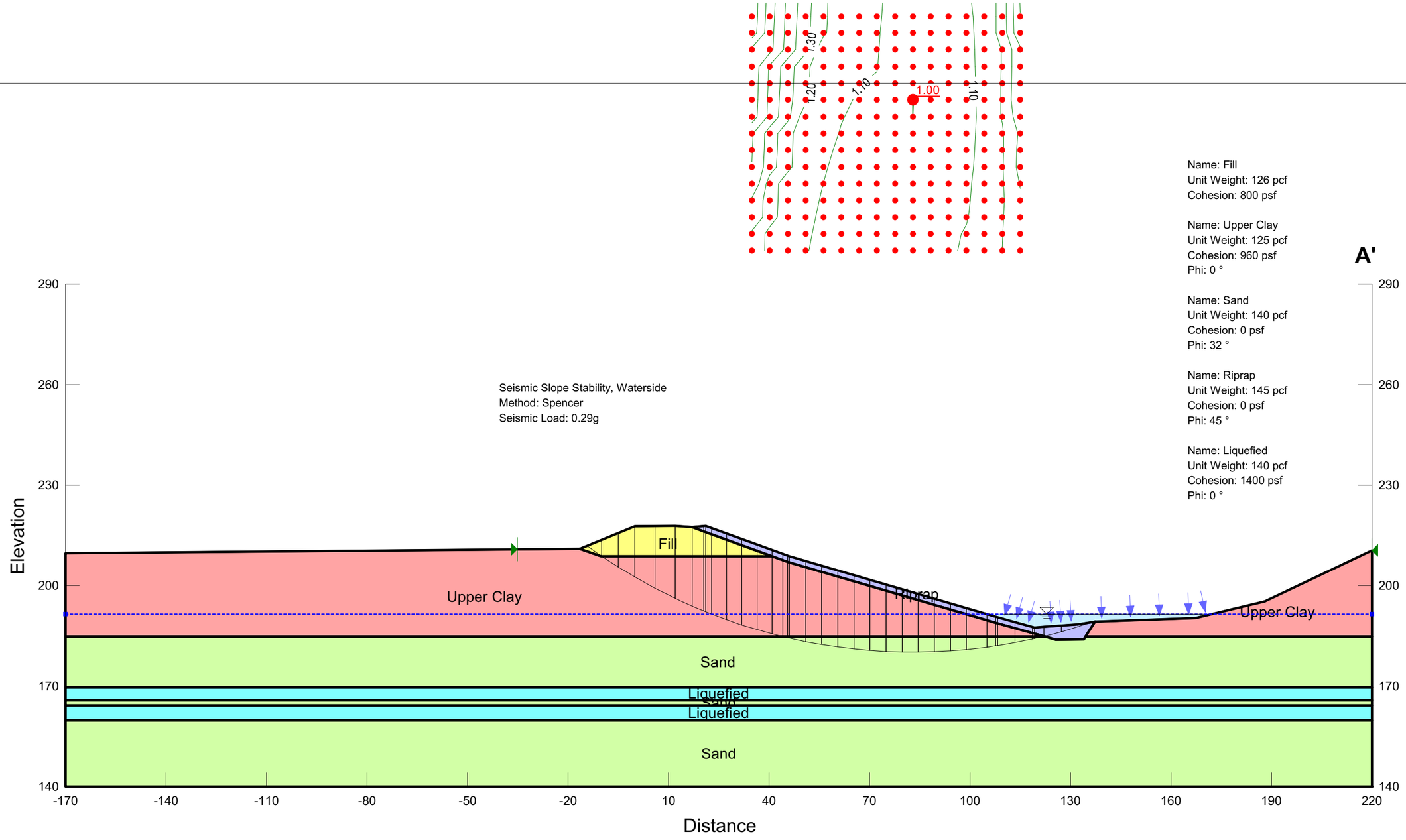


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|---|---|---|---|---|---|---|
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Waterside<br>Cross Section A<br>Levee Station 23+46 | **Figure**<br>14-4 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/7/2009 | Checked | | | |

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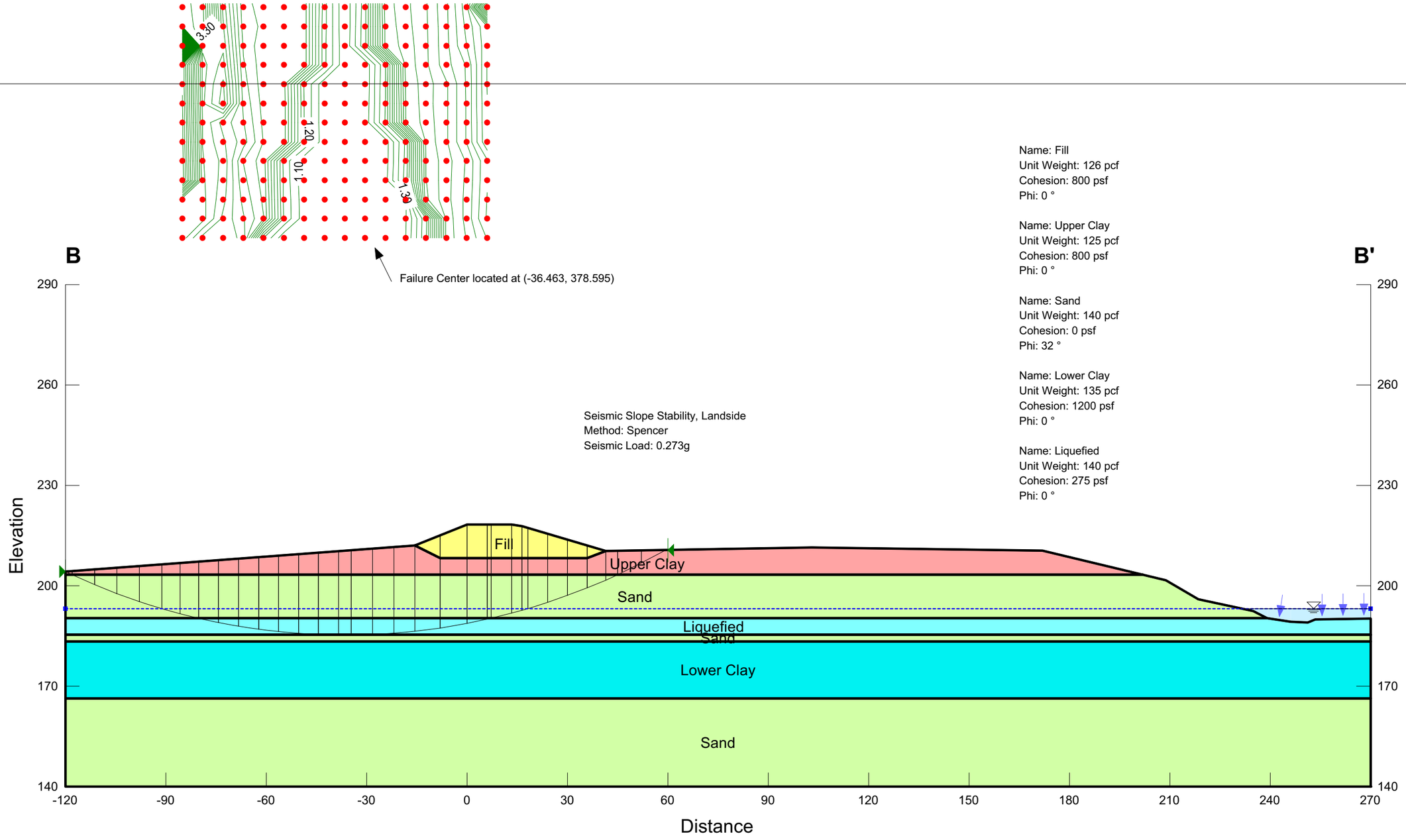


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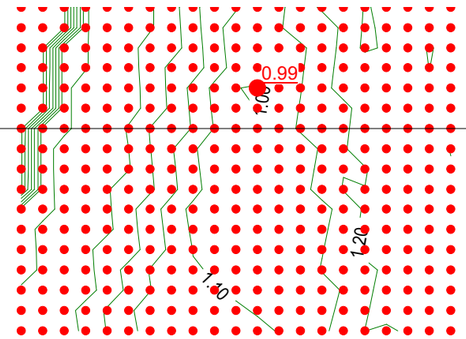
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Landside<br>Cross Section B<br>Levee Station 31+69 | **Figure**<br>14-5 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/6/2009 | Checked | | | |

Directory: I:\Project\13000\13815.001\12000 CALC EVAL ANALYSES\Geo Studio\Section B\ B_RS_22916_SeismicKyLiq.gsz Date: 4/6/2009 Time: 11:50:46 AM Last Edited By: Levulett, Darci



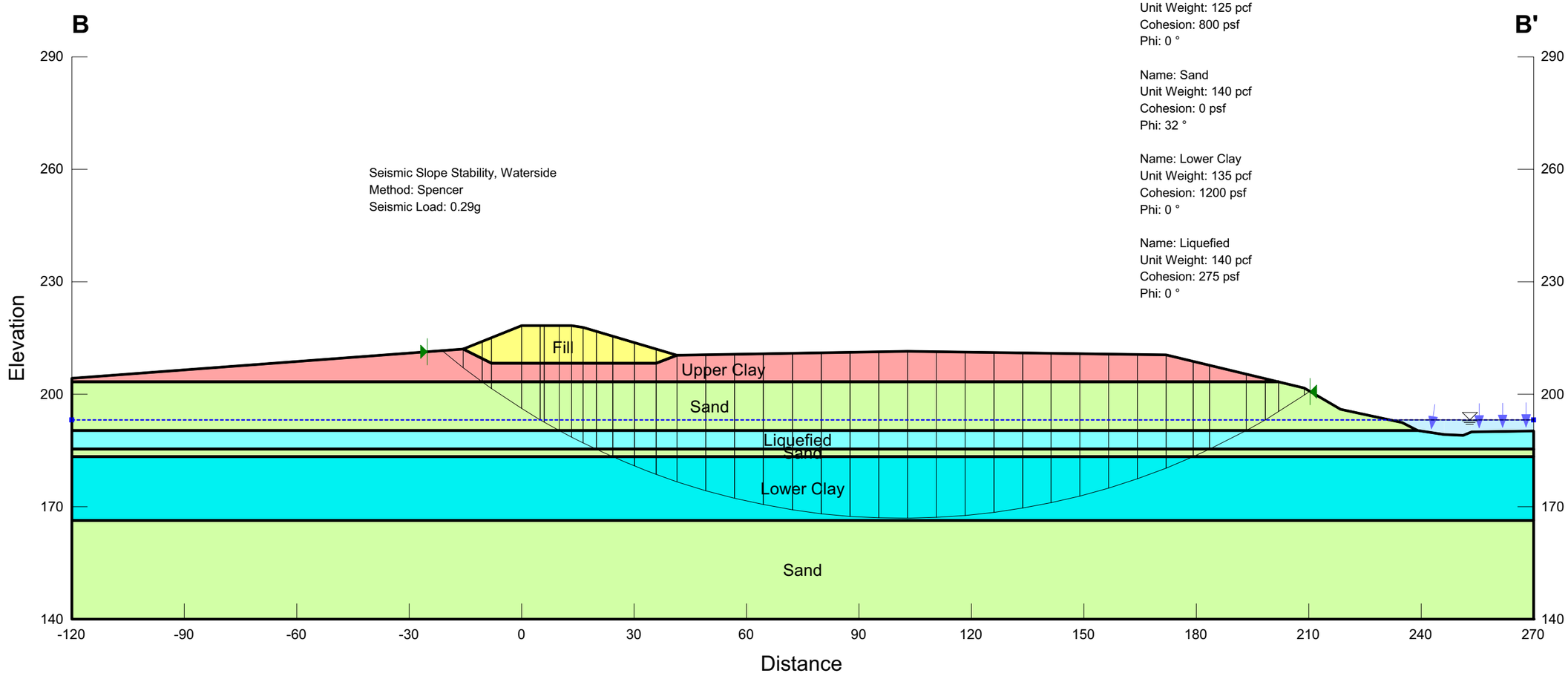

Name: Fill
Unit Weight: 126 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Upper Clay
Unit Weight: 125 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Sand
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 32 °

Name: Lower Clay
Unit Weight: 135 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Liquefied
Unit Weight: 140 pcf
Cohesion: 275 psf
Phi: 0 °

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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Waterside<br>Cross Section B<br>Levee Station 31+69 | **Figure**<br>14-6 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/6/2009 | Checked | | | |

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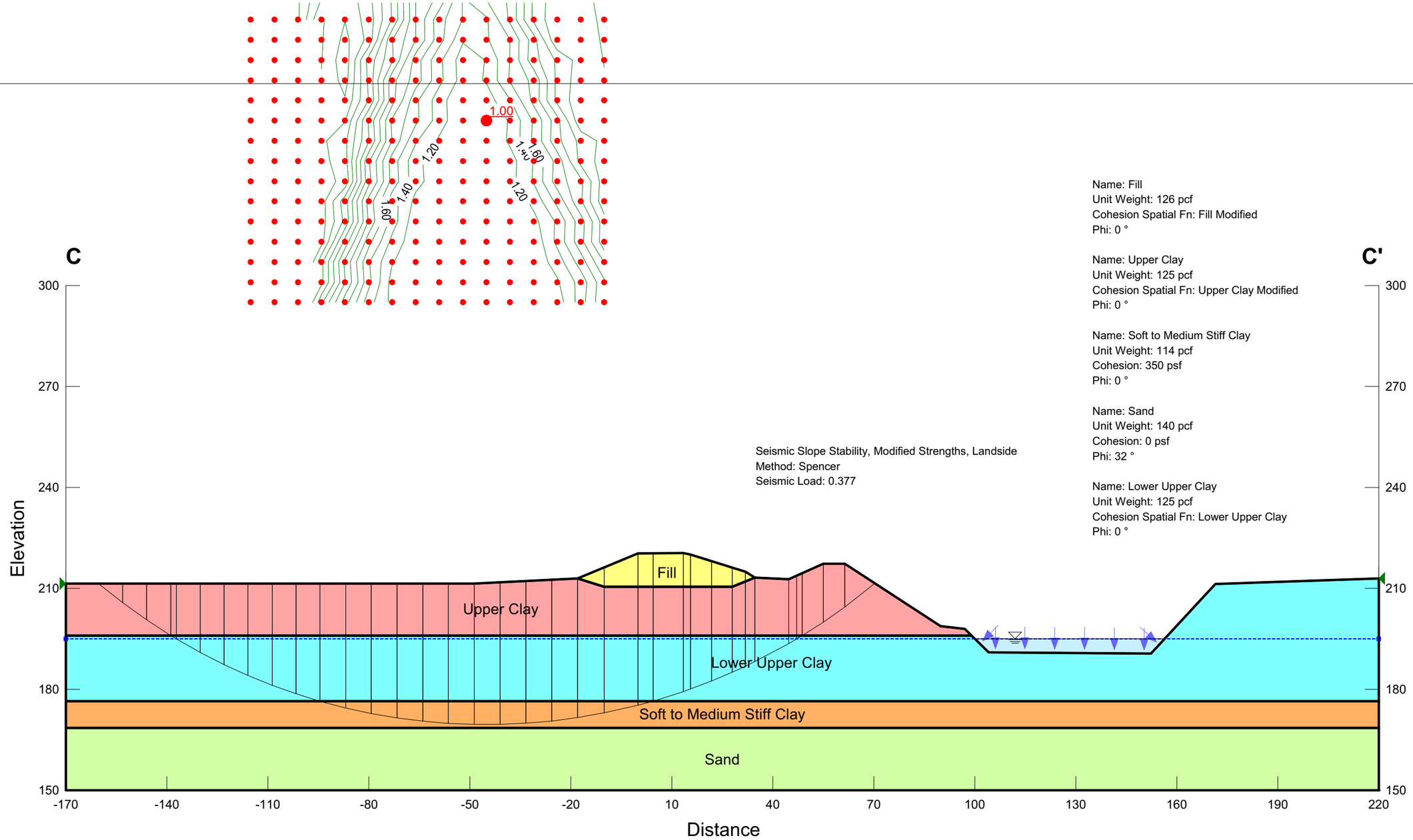


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Landside<br>Cross Section C<br>Levee Station 48+16 | **Figure**<br>14-7 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

Directory: I:\Project\13000\13815.001\12000 CALC EVAL ANALYSES\Geo Studio\Section C\ C_RS_24563_Seismic_Ky_Block.gsz Date: 5/19/2009 Time: 11:34:41 AM Last Edited By: Levulett

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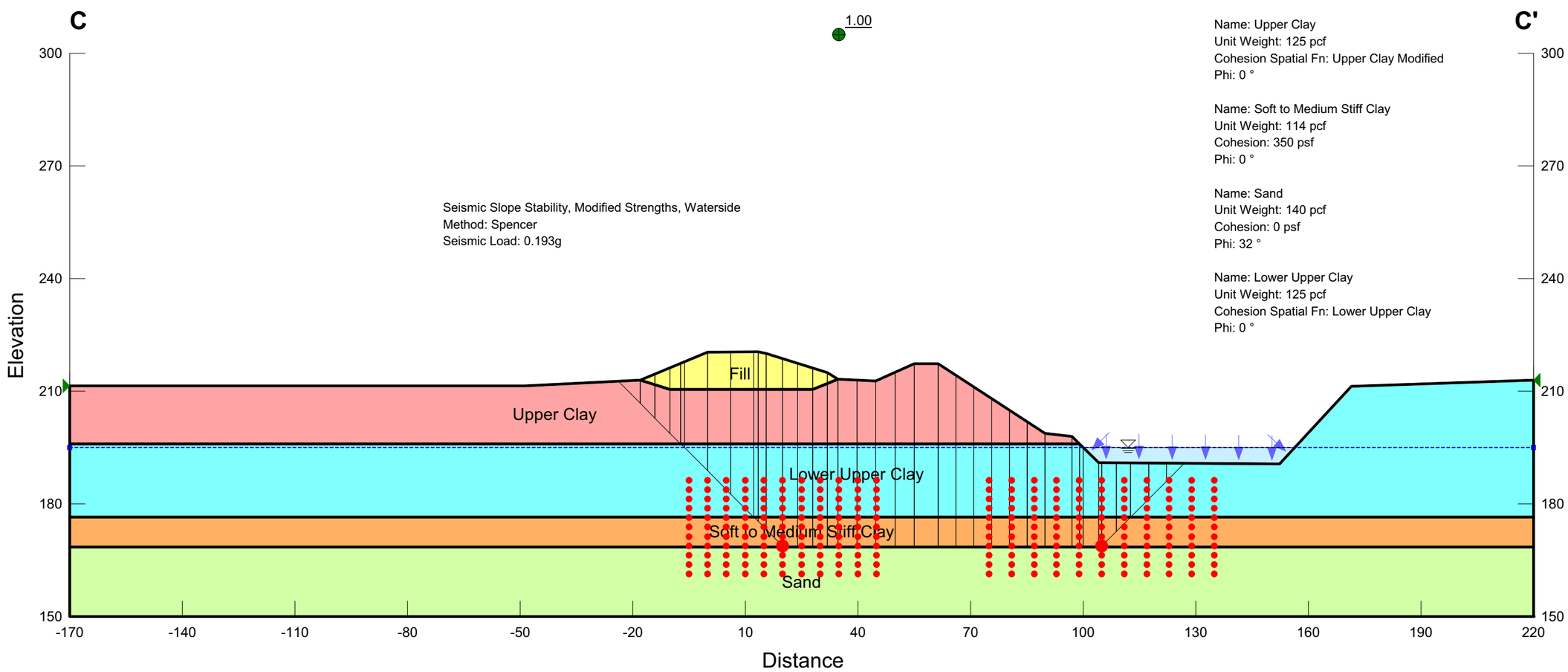


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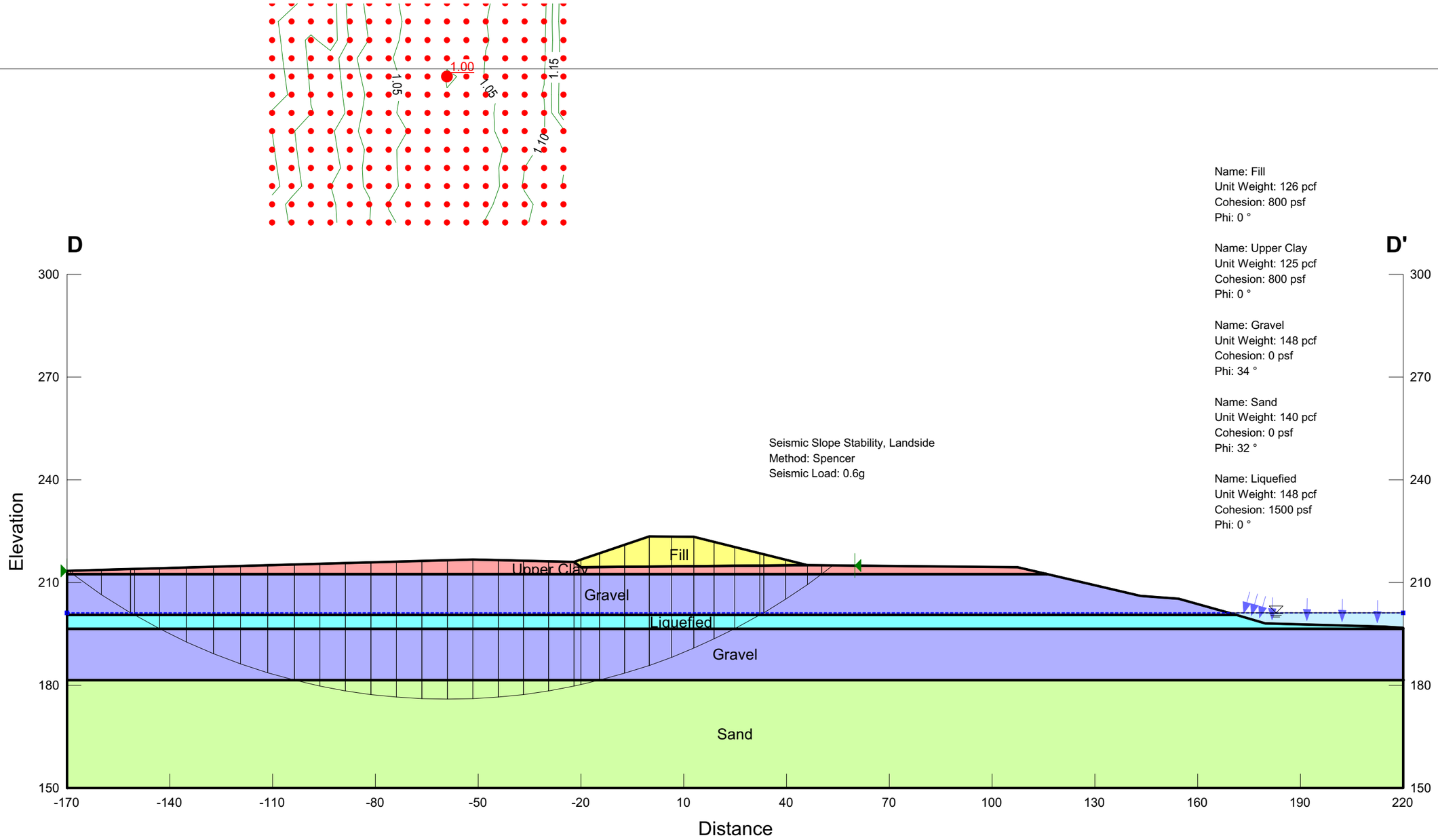
| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Waterside<br>Cross Section C<br>Levee Station 48+16 | **Figure**<br>14-8 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/19/2009 | Checked | | | |

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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Landside<br>Cross Section D<br>Levee Station 72+62 | **Figure**<br>14-9 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/6/2009 | Checked | | | |

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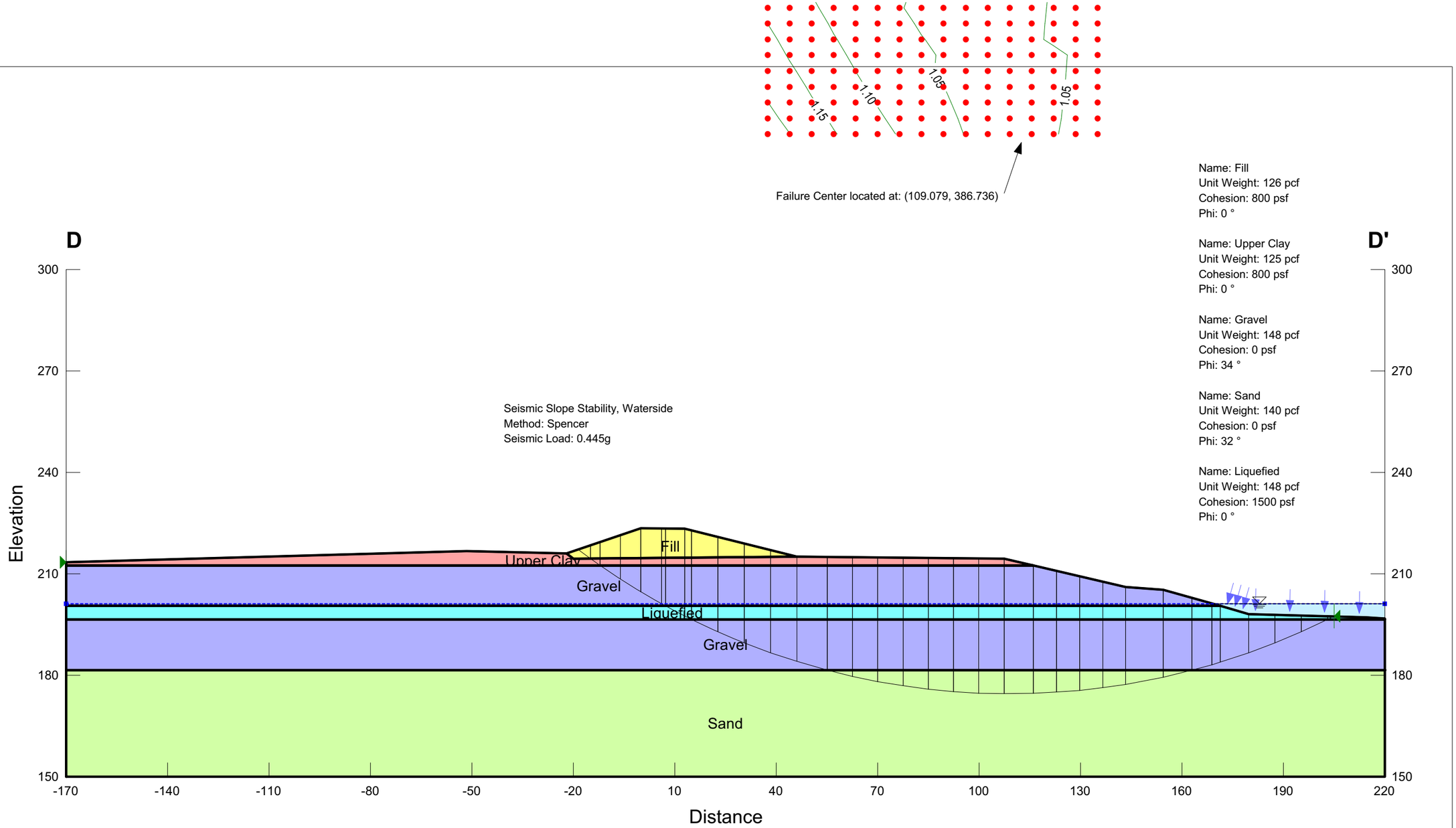


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Waterside<br>Cross Section D<br>Levee Station 72+62 | **Figure**<br>14-10 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>5/6/2009 | Checked | | | |

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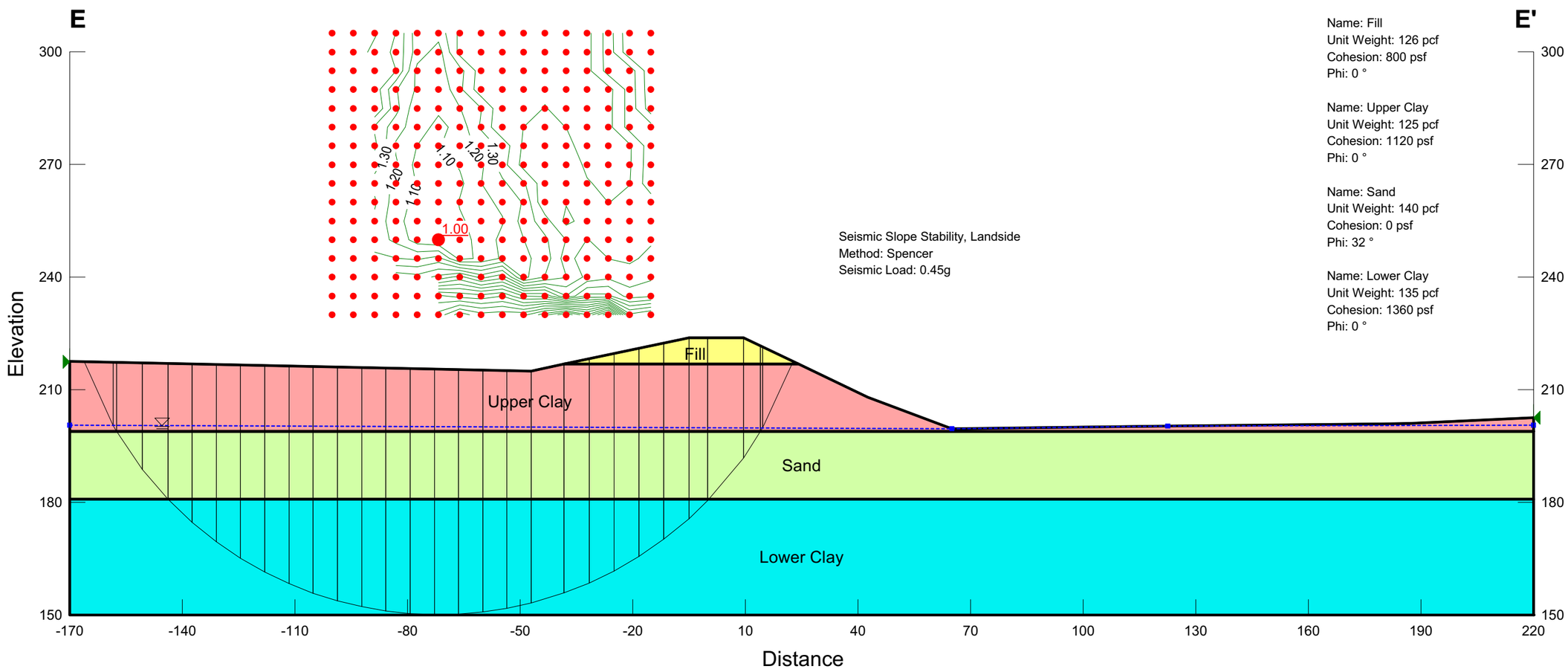


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Landside<br>Cross Section E<br>Levee Station 90+91 | **Figure**<br>14-11 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/7/2009 | Checked | | | |

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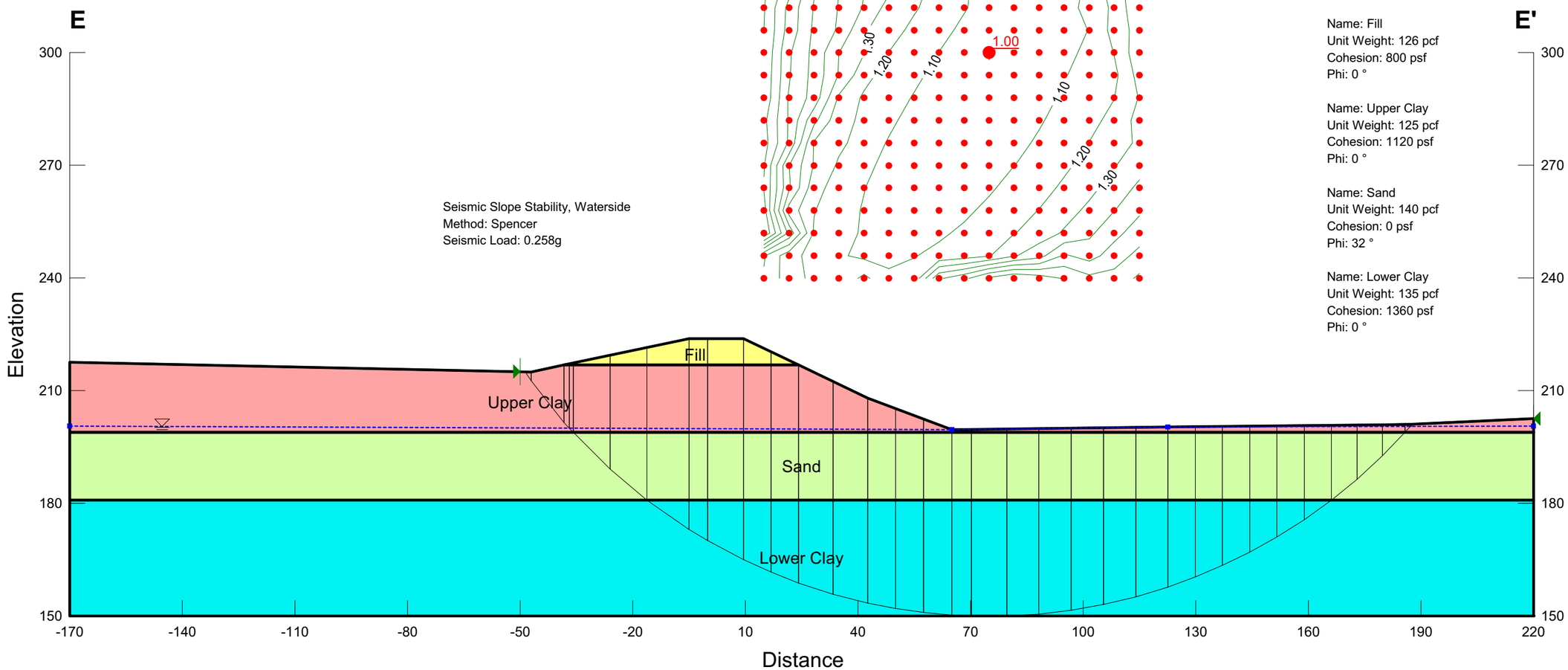


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| amec | **Scale**<br>1 inch = 30 feet | Project Number<br>13815.001 | Drawn<br>D. Levulett | **Uvas Creek**<br>**FEMA Levee Recertification**<br>**Gilroy, California** | **Seismic Yield Determination**<br>Waterside<br>Cross Section E<br>Levee Station 90+91 | **Figure**<br>14-12 |
| | | Task<br>0A.03 | Analyzed<br>D. Levulett | | | |
| | | Date<br>4/6/2009 | Checked | | | |